I

BEHAVIOUR OF SOILCRETE UNDER SHORT AND LONG TIME LOADINGS

BY

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MAY, 1990.

THESIS DECLARATION

I hereby declare that this thesis has been composed by myself and that it is a record of my own research. It has not been accepted in any previous publication for a higher degree.

All quotations are distinguished by quotation marks and the sources of information are specifically acknowledged by means of references.

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THESIS APPROVAL

We hereby recommend that the thesis prepared under our supervision by MR. STEPHEN PINDER EJEH entitled "Behaviour of Soilcrete nder Short and Long Time Loading" be accepted in fulfilment of the requirement for the award of DOCTOR OF PHILOSOPHY in (Civil Engineering) of Ahmadu Bello University, Zaria.

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DEDICATION

This thesis is fully dedicated to Eunice, David and Almighty God ,the father of the Lord Jesus Christ.

STEPHEN PINDER EJEH.

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NOTATION

Φ_∞	=	Creep ratio
C_∞	=	Creep at infinity
C_i	=	elastic deformation on loading
E_C	=	modulus of elasticity in GPa
ρ	=	density of concrete Kg/m^3
f_{cu}	=	cube strength N/mm^2
f_{cyl}	=	specific characteristic cylinder strength N/mm^2
σ	=	stress N/mm^2
σ_{max}	=	maximum stress
ϵ	=	strain
ϵ_{max}	=	max strain
t	=	time in days
C_{sp}	=	specific creep

ABSTRACT

Based on the Rheological properties of concrete materials, soilcrete rheological properties were carefully examined. It was observed that both concrete and soilcrete materials have the same rheological behaviour. In real quantitative deformational values, soilcrete materials appear to have more pronounced values when compared to concrete materials. However, the mode, mechanism, and shape of deformational - time graphs are the same for both materials.

CHAPTER ONE

INTRODUCTION

The importance of researching and developing a building material that is abundant in any country cannot be over-emphasized. Soilcrete materials are mainly stabilized soil. The soil materials are readily abundant in Nigeria. It is in A-6 plastic clay soil group in accordance with United States Bureau of Public Road Classification system.

Soilcrete rheological properties is the study of the relationship between stress and strain within the soilcrete materials and other physical and physico-chemical properties that may contribute to this relationship. It also involves the study of the physical and physico-chemical properties of the main material that constitute soilcrete material.

In this thesis, an intensive literature review revealing conflicting research reports of various investigators were reported and possible reasons for the conflict given were necessary and possible. This is coupled with various physical and physico-chemical tests and analysis on both Lateritic and Soilcrete material. The short and long-term deformation tests, development of strength over two years and a good method of curing for soilcrete materials were determined. This lead to analyses of test results, observations, conclusions and recommendations for further research areas in soilcrete and related materials.

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CHAPTER TWO

2.0 LITERATURE REVIEW

2.1.0 Lateritic Materials

2.1.1 Definition of Laterite

In engineering practice, the term laterite is used in a broad sense, denoting a wide range of highly weathered trapical and subtropical soils with colours varying from light grey over yellow to dark red. The term lateritic soil is the one most commonly used, but in some countries other local names are used. Some definitions of lateritic soils are based on colour, particle size distribution while other definitions are based on chemical analysis, physical and structural appearance and morphology. Ayetey et al /26/ defined laterite as a hardened or soft weathered product of tropical and sub-tropical weathering which is leached of bases but enriched in sesquioxides of iron and alumina, in the form of clay minerals especially kaolinites and secondary iron minerals such as geothite, limonites and hematite. Other authors /146/ puts it gthat lateritic materials include a large succession of reddish, tropically weathered, decomposition products starting from fresh rock and ends with sesquioxide-rich pedogentic-rock. Pendelton and Sharasuvana /300/ defined laterite as soils in which laterite horison is found and lateritic soils as those in which there is an immature laterite composition from which a true laterite

horizon will develop if appropriate conditions prevail long enough/ 300/.

Some authors like Nixon and Skipp/287/felt that the word laterite should only refer to the concretionery coarse materials and fine grained materials as clays or latosols while Nanda and Krishnomachari/267/ though believes in Nixon and Skipp to some extent but preferred the word to be the massive concretionery rock-like materials also known as cuirasses. Adepegba, Nanda/4, 268/ explained that the word laterite was given by Buchanan in 1800 to surface formation with cellular concretions in Southern India. The word was derived from a Latin word "Later" which means bricks. Both aggreed that laterite is a weathered product rich in secondary iron, alumina and both. However, Ola /291/ felt that the ratios of silica to sesquioxide $S_iO_2/(Fe_2O_3 + Al_2O_3)$ should be the best way to define laterite. He suggest that silica to sesquioxide ratio of less than 1.33 are indicative of laterites, while 1.33 to 2.00 is of lateritic soil and those greater than 2.00 is of non-lateritic types.
He however, used a local terminology similar to Nixon and Skipp since it is not convient from engineering point of view particularly where there is a lack of adequate laboratory facilities to use the ratio. Generally the authors tends to agree that lateritic materials are weathered product and tend to be strongly coloured by iron compounds.

However, Gidigasu /153/ classified the definition of laterite into that which is based on hardening property, chemical composition and morphological propoerty. He also strongly believed like the others that laterite is weathered material which is mainly a ferruginous, vesicular, unstratified and porous materials with yellow ochres due to high content of iron.

This review of the definitions of laterites reveals how complex lateritic soils are, apparently defying any satisfactory geological, chemical or pedological definition. While it is important in many respects to recognize the difference between lateritic and non-lateritic soils and further to recognize the differences between the various type of laterites, the fact remains, that the true consideration for identification and evaluation of lateritic soils for engineering purposes is not "what is its name" but what are its significant geotechnical characteristic and engineering behaviour.

2.1. 2 Location of laterite Materials.

The location and distribution of lateritic materials has been associated with temperature and rainfall conditions that characterise the earth's surface between latitude 35° N and 35° S /301/.

In addition to those places where they have been found, they may also be expected at locations where though they have not yet been discovered, there are indication of environmental conditions suitable for their formation. The world map shows the distribution and location of lateritic materials. Thus these materials can be found in Australia, Africa, America, and Asia as shown in A-1

Established procedures for locating natural soils have been found applicable to lateritic materials / 301, 379, 388 /. To cut down costs and at the same time accelarate the data collection of lateritic materials, considerable use have been made of aerial photography and sounding technique of soil-survey procedures in locating laterite / 119, 196, 237 /. Agricultural soil-survey maps, pedological report, and some geological maps have also been used extensively for locating lateritic materials / 87, 153/. Geophysical Method for locating materials in the field had also been used. However the development of airphoto interpretation techniques for bounding areas of soils of similar

engineering characteristics for making engineering soil maps led to the rather wide use of this tool in the location of lateritic materials / 264, 237, 301, 246 /

2.1.3 Choice of Laterite

Lateritic soils range in performance from excellent to poor. It is therefore necessary to know ways of choosing suitable laterite for engineering purposes. In this aspect, various authors/researchers (Ayetey and Castle /26/ ; Bani /29, 30/De Grast-Johnson /109/ ; De Graft-Johnson et. el./ 110/; Gidigasu /147/) carried out investigations on different paramenters of soil and have established an upper value for each parameter as regards the suitability of lateritic materials.
The word suitability here implies that lateritic materials are suitable for engineering purposes. The summary of various work by different authors is as shown below -

- RANGE OF PARAMETER VALUES FOR SUITABLE LATERITE

Parameters		Suitable Lateritic Materials	Non-suitable Lateritic materials
1.	Liquid Limits	Less than 50	greater than 50
2.	Plasticity Index	Less than 20	greater than 20
3.	Colloidal Activity	Less than 1.0	greater than 1.0
4.	Linear Shrinkage (%)	Less or equal 5.0	greater than 5.0
5.	Aggregation Index	Less or equal 1.0	greater than 1.0

6.	Potential Swell (%)	Less or equal 10	greater than 10
7.	Montimorrillonite Content	Low	high
8.	Base Exchange	Low	high
9.	Calcium Carbonate Content	Lacks $CaCo_3$	Contains $CaCo_3$
10.	Flakiness and Elongation	Low	High

Although these parameters are useful in identifying lateritic samples, some of them could be misleading if used alone. The clay-size content and plasticity index vary with sample preparations and testing procedures. Gidigasu /147/ pointed out that the troublesome lateritic materials appear to be associated with region of recent volcanic activity and continously wet climate with rainfall distribution above 60 ins (1620mm) per year. The materials are highly hydrated and with low field density (20-70 lbs/ft^3) (3.14 - 11.0 KN/m^3). It is usually sensitive to drying and remoulding during index tests and this hinders reproducibility of test results by standard laboratory testing procedures. Bhatia and Hammond /64,170after their investigation were able to rate the lateritic materials from excellent to poor. Investigations were carried out on four different parameters and recommended possible values for the respective type of lateritic materials. This is outlined below.

PERFORMANCE RATING OF LATERITIC MATERIALS

Rating of the Lateritic materials ⇒	Excellent	Good	Fair	Poor
Los Angelos Abrasion value (%)	< 40	40-50	50-60	> 60
Aggregate Impact value (%)	< 30	30-40	40-50	> 50
Water Absorption (%)	< 4	4-6	6-8	> 8
Specific Gravity	> 2.85	2.85-2.75	2.75-2.58	< 2.58

The specific gravity of lateritic materials is a function of the state and concentration of iron and the degree of laterisation. I-dentification and evaluation of lateritic materials with regard to genesis, degree of decomposition, laterisation or dessication may be misleading and lead to erroneous conclusion. The significant properties which influence the performance are specific gravity, water absorption and aggregate strength though aggregate strength on its own alone is not reliable. Using all the values in conjunction with each other as outlined is very useful in choosing a good lateritic materials.

Wide variations in particles size distribution and plasticity test results with differences in the degree of pretest drying, type of dispersing agents, time of mixing and removal of free iron oxides during the tests have been widly reported by many investgators for

problem lateritic soils. Mineralogically, problem lateritic soils generally contains the clay-mineral halloysite, and also known to contain hydrated iron and aluminium oxides, some of which change irreversibly to dehydrated forms on drying or desiccation. Investigations into these problem lateritic soil show that they have low compressibility, highly permeable and possess high angles of shearing resistance /193, 153/.

The problem lateritic materials also have friable or granular structure which appears to be derived from the presence of sesquioxides, especially free iron oxides which are adsorbed on the surfaces of the clay minerals, coat the pore walls and/or fill the voids and bind the indigenous soil particles into tiny spherical or spongy aggregations /153/.

However, secondary minerals which are most susceptible to changes in properties with drying and are found to be in problem lateritic materials have been identified as hydrated holloysite, geothite, gibbsite, allophane /153/.

It has also been suggested that if all the iron in the lateritic materials occur in a ferric state, and no factor is present to promote a reducing environment, the lateritic material (soil) is essentially stable and no change can be expected but if however, all the iron oxide does not occur in the ferric state, or

if conditions prevailing will promote a reducing environment allowing further mobilization of iron, then one could expect changes and the lateritic material is not stable.

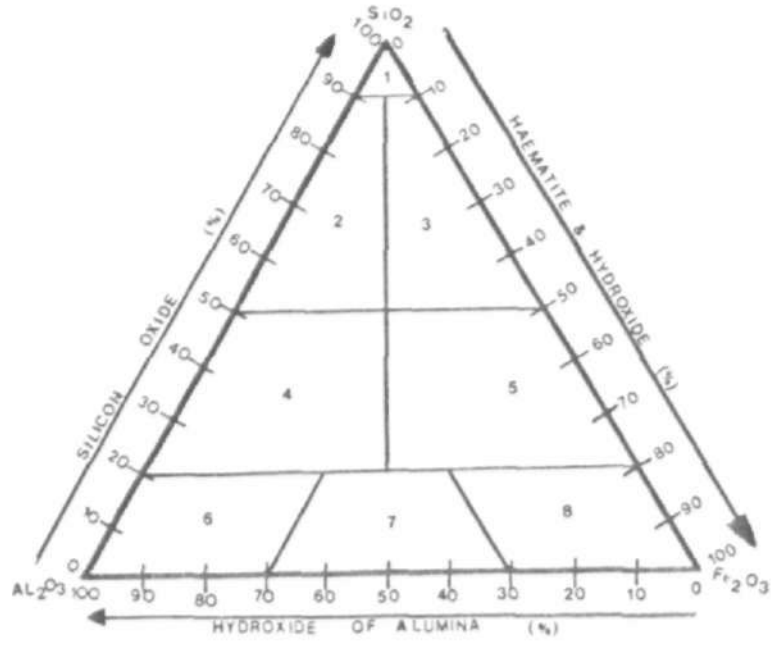
Finally, it appears from literature review that at present, it is difficult to develop a complete satisfactory criterion for the identification of problem lateritic soils. This is because the available data are imprecise and the testing methods have not been internationally standardized.

2.1.4 Classification of Laterite

Lateritic materials vary greatly in their texture, compositions and occurrence especially those of the tropics. Thus, the need for its classification cannot be over-emphasized. However, any attempt at classification must avoid over simplication and at the same time be of value to the engineers especially, who use laterite as a material for construction. Many authors /147, 148, 149, 150, 153, 154, 371 / have come to the conclusion that the classification methods for temperate lateritic materials which is based on their mechanical behaviour AASHO (mechanical and chemical factors) and USSCS (primary agricultural system) do not give satisfactory results with the tropical lateritic materials. The AASHO and the USSCS both points on measurements of relative proportions of fine and coarse textural materials and Classify the materials into sand, clay and organic matter. Even though the methods were very successfull for temperate lateritic materials, it is not

successfull with the tropical materials, as such, additional factors like age, drainage, climate, colloidal silica content, alumina-iron ratio and silica-sesquioxide ratio has to be invoked to achieve satisfactory classification. Others base their classification on pedogenic characteristics, morphological characteristic, topography and drainage conditions, and climatic vegetational conditions. Laboratory studies backed with field experience in many African countries reveal that none of these methods on its own isolated form give satisfactory results for tropical materials.

Classification as suggested by Konta in Gidigasu /153/ and later modified is as shown below.



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. Textural classification of lateritic materials.

Key

1. - Clay (residum) with lateritic admixture
2. - Lateritic clayey (residum)

3. - Ferruginous lateritic clay (residum)

4. - Clayey laterite

5. - Clayey ferruginous laterite

6. - Aluminous laterite

7. - Laterite

8. - Ferruginous laterite.

When field examination of lateritic materials show that SiO_2 values are high, which can be attributed to free quartz and combined silica, thus giving quartz as the predominant mineral, sand was substituted for clay in the classification. Various other authors (U.S. Burrau of Soils, U.S. Engineer department, U,S. Department of Agriculture, Gidigasu/147/ ;) have proposed textural soil-classifications purely on the basis of their grain-size distribution. This type of classification dates back to time when it was not yet realized that physical properties of fine-grained soils, with similar particle-size distribution, can be widely different. The simplicity of textural soil-classification systems and the fact that they can be applied with little experience are the reasons why they are still widely used. Original textural classifications were based on the relative percentages of the sand, silt and clay-size fractions. They are facilitated by plotting these three fractions on a triangular chart. The difficulty of all triangle classifications, however, lies in the

fact that those physical characteristics of fine-grained soils which are of interest to the Civil Engineers are often not reflected by the three fractions alone. However, textural classifications based on the entire range of particle sizes may be useful for cohessionless soils of the same geologic origin.

Researchers / 109, 110, 147 / having studied about 650 samples of lateritic materials on the other hand grouped the materials in Ghana into four as follows:-

1. Modular or concretionery laterite
2. Iron stone hard pans or cap rock
3. Ground water laterites with detrital quartz
4. Colluvials and terrace of laterite.

The manner of grouping is useful especially in the preliminary selection of laterites in the field since each one of the group has specific range of physical properties or characteristics.

Using the process of laterisation, Gidigasu /145, 152/ grouped lateritic materials texturally into three main groups. These include:-

1. Lateritic rocks and boulders-mostly concretionery lateritic rocks and rock pieces.
2. Lateritic and lateritic quartz gravels-most naturally occuring gravels.
3. Lateritic fine-grained materials.

The above three samples reflects definite stages in the process of laterisation. D'hoore / 117/ based his

classification of laterite on the amount of annual rainfall. He divided laterite into groups as follow:-

1. Ferruginous laterite -occurs in areas with rainfall between 20 to 60" (508-1524mm) and liquid and plastic limits less than 50 and 25 respectively.
2. Ferralitic or Ferrisol laterite - occurs where rainfall is above 1524mm and have liquid limits and plastic limits above 50 and 25 respectively. D'Hoore classified lateritic materials above using his pedologic map of Africa.

In actual sense the classification is based on pedological and rainfall factor. Ajayi /7/ in his own research as a foundation Engineer classified lateritic materials into three major classes using grain-sized distribution. Those classes include:- (1) Very coarse lateritic materials, (2) Coarse lateritic materials, (3) Fine lateritic materials. This method of classification is exactly the same as that proposed by Gidigasu / 145, 152 / on Ghanain soil.

Nanda /268/ seems to have a different view on ways of classifying lateritic materials. He classified lateritic into two broad varieties namely :-

(1) Residual laterite - which is believed to be formed in-situ under free drainage conditions.
(2) Transported laterite - which is the moulded product of old indurated layers that have been broken up.

This in-situ (surface) lateritic materials according to Nanda are usually between 0.1 to 1.0m below the surface.

It is important to always remember that soils formed under similar weathering conditions are known to have simlar morphological, mineralogical and geotechnical characteristics. Consequently , attempts have been made as proposed above by Nanda and also by other authors / 153, 147 /, who have attempted to develope simple classification system for some laterite based upon the relationship between the soil, their genesis and pedogenic factors. Thus significant correlation have been reported between the parent-rock types and the plasticity classifications of various lateritic materials.

Another usefull methcd of classifying tropically weathered soils (lateritic materials) could be on the basis of the relationship between the genetic soil type and their position on the Casagrande plasticity chart. Even it was found that lateritic clay materials of volcanic origin generally plot below the A-line while other red and black lateritic clay materials formed over igneous basic parent rocks plot above A-line. However other studies /110, 153 /have revealed that no unique relationship between the genetic origin and the plasticity characteristics may be obtained for highly laterised soils and quartz-laterised materials mixes. For example, it has not be possible, in terms of the plasticity classification of the fines, to distinquish between the concretionary and quartz

materials; this is because the influence of the content of fines in concretionary - quartz material on the plasticity characteristics may not be unique.

The critical condition on which the validity of each standard textural and plasticity of lateritic materials classification system is that both the particle-size distribution and plasticity test results should be reproducible using the standard laboratory procedures. This condition is met by most lateritic materials whose different mode of formation, colour, mineralogical composition, morphology, etc. do not necessary mean wide differences in engineering properties beyond the scope of presently used test methods. Laboratory studies supported by field experience have shown that laboratory tests carried out on these materials using different pretest preparations and testing procedures lead to essentially the same results /157/.
Normal lateritic materials may thus be identified on the basis of standard soil-classification properties (i.e. particle-size analysis and Atterberg-limit tests.

Many authors have various ways of classifying lateritic materials but one fact is always evident and that is the method used for temperate materials is not the best for the tropical meterials.

2.2.0. STABILIZING AGENTS

2.2.1. HISTORICAL NOTE

Soil stabilization has been practised in many countries for many centuries. Stabilisers such as natural oils, plant juices, animal dug and crushed antihills (386) has been used for many centuries. In Nigeria, stabilization of laterite can be traced as far back as 1681 A.D. The Katsina Gobarau minerate is an example. It was believed to have been built at about 1671 A.D and later rebuilt at about 1921 A.D. Habe dynasty was said to have built it. It has a plan area of about 68 m^2 and about fifteen metres high. The material used was principally laterite which is mixed (stabilised) with he following materials.

1) Loda - a syrup prepared from a tree called loda
2) Katsi - mostly waste of dyne
3) Gashin - jima - mostly horse hair.

The three materials are mixed in a local ratio of two bags of katsi to two bundles of loda (dried) to one bag of Gashim jima. This proportioned materials are mixed with laterite. These materials are also used as finishes and have been believed to last for two years before applying another finish.

The minerate in Katsina is still standing up to date. In the northern part of Nigeria, animal dung, grasses (thatches) and ground husk of locust beans' fruit (Makuba) and rice husk have been in use for years back. Usually the laterite is dug and the animal dung/grass with water is poured on it and allow to stay for a week before mixing using water. After mixing, it is then used for blocks or mud-wall.

The history of lime stabilisation dates back to 1866 when Van Derburgh in England patented the process of hardening sand-lime bricks /31, 32 /by steam under pressure. This was further developed and patented by W. Michaelis of Germany in 1880. The earliest commercial development took place in Germany in 1898 where the bricks were manufactured in industry. The industry advanced slowly initially but more rapidly in the early part of the twentieth century.

2.2.2. PORTLAND CEMENT

Lateritic materials used for building purposes are often in need of stabilisation due to their low wet strength and tendency to

degrade on working. The commonest stabilisers are cement, lime or a mixture of the two.

Portland cement is a technologically sophisticated material. Raw material purity is crucial to the end product and attempts to produce it in a small scale from low quality minerals have not been widely successful. In stabilising soil using cement, the aim is to produce in the soil the following.

(a) maximum dry and wet compressive strength
(b) adequate cohesion
(c) minimal swelling, and low affinity for water
(d) resistance to erosion by wind, rain, running water and surface abrasion.
(e) high shear strength
(f) low porosity

The mechanisms by which lateritic materials react with cement to achieve these aims vary and is discussed below.

CEMENTATION

The cementing action is produced by deposition within the pores of the lateritic

materials of an insoluble binder capable of embedding the lateritic material particles into a matrix of cementitious gel. The binder or gel matrix may be formed by hydrating the cement, or by the reaction of the cement with some other material present. The materials that may be present include precipitated lime and other mineral salts. Cementation requires three processes to occur /386/. These include formation of gel, distribution of the gel throughout the lateritic materials and the binding of the gel to the lateritic materials. Portland cement gel is formed by the irreversible hydration of a complete mixture consisting principally of calcium silicates and calcium aluminates, creating a network of interlocking fibres of crystalline alluminosilicate around cores of un-reacted particles of cement. This fibrous gel, weak and amorphous at first but becoming strong and crystalline and fills the pore space of the lateritic material and binds the particles together. The hydration of the various component which is temperature dependent, proceeds at different rates. Tricalcium aluminate occurs almost instantaneously at 20^{o}C and is responsible for early strength gains, while tricalcium silicate takes several hours at the same

temperature and is more important in controlling the final strength of stabilised soil. Curing is irreversible, and it is essential that moulding or compaction takes place soon after mixing, otherwise the newly formed gel is broken up and part of the strength is lost.

The formation of the gel by the portland cement is essentially independent of the lateritic materials' composition. The precipitated lime present also causes its own cementation in addition to that formed by hydration. This cementation, by contrast, depends on the ability of the precipitated lime to attack materials present in the laterite, forming in the pores of the laterite an insoluble colloidal gel similar to that produced by the hydration process. Clay soils, with their aluminosilicate minerals and varying quantities of amorphous or partly crystalline silica, alumina and iron hydroxides, are especially suited to such a cementation reaction.

The organic matter content in the lateritic material hinders the development of high strength / 85, 271 /. However, studies on typical lateritic materials from different climatic-vegetational conditions reveal that there is no

straight forward correlation between the strength gain and the organic matter content; rather other factors are involved. The importance of the clay mineralogical assemblage in soils stabilisation was emphasized by various investigators /95, 96, 124, 153/. Croft /96/ after his investigation on the influence of clay mineralogy on soil stabilisation reach the following conclusions:

(a) Kaolinitic and illitic lateritic materials are more suitable for cement stabilisation than one with large amount of expansive clay minerals. The amount of clay present in a soil will determine the quantity of cement required.

(b) Heavy clay minerals are difficult to stabilise with cement, since large quantities may be required. If the predominant clay mineral is an expansive montmorillonite and if present in an amount large enough to control the behaviour of the soil, then cement may prove uneconomical and lime may be used.

The addition of cement as stabiliser to lateritic materials tend to decrease the maximum dry density and increase the optimum moisture content of the lateritic materials. Addition of 6 - 8 per cent cement by weight is found to decrease optimum density by 2 - 4 per cent and increase optimum moisture content by 2 - 3 per cent / 386 /. The increase in confined compressive strength for laterite - cement mixes cured for 7 - 28 days is found to be in the range of 6 - 12 per cent per lb/ft^3 (0.38 - 0.75 per cent per kg/m^3). The effect of cement is to collect the clay fraction in laterites and thus increase the plastic limit and decrease the plasticity index while enhancing soil binding activity /96, 386/.

The time lapse between mixing and compaction of cement - stabilised lateritic materials can seriously affect the strength, as hydration of cement begins as soon as moisture is added to the mix. West /398/ has reported a loss of between 50 and 60 per cent strength after a time lapse of about two hours between mixing and compaction time, while Daniel /99/ has reported a loss of 13 - 15 per cent strength for lateritic gravel material after a time lapse of 50 min. Experience has shown

that the time interval should not be more than twenty minutes for a better result. Curing temperature and age markedly affect the development of strength in stabilised soil-cement; strength increases with curing and age at a rate which depends on the soil characteristics and cement concentration. These effects are of significance in tropical buildings where unconfined compressive strength at 7 - days is typically 75 - 85 per cent of its 28 - day value and in the temperate climate the 7-day unconfined compressive strength is about 45 - 55 per cent of the 28 - day value.

Experience in the cement stabilisation of lateritic soil materials suggest that the stabiliser requirements may not always be judged from the physical properties of the lateritic materials alone, but the overall genetic and compositional factors may be considered simultaneously to arrive at a rational procedure that may predict their reaction to stabilisation and also predict field performance quite precisely /153/.

Other stabilizing agents that is worth mentioning inlude lime and asphalt /386, 153, 156,/, bituminous materials, cut -back asphalts, road tars and asphait emusion for soil stabilization /405 / .

2.2.3. LOCAL STABILISERS

As noted in the historical note, soil stabilisation has been practised in many countries for many centuries by using the available local materials as their stabilising agent. Much has been said by the use of Iodo, Katsi, Dashim-jima, husk of locust beans and trampling vegetable matter into wet lateritic mud followed by fermentation as local stabilisers in Nigeria. This tampling vegetable matter into wet lateritic mud, followed by fermentation has been found to produce a hard material suitable for building and believed to have been bound by ferruginous cements.

Animal dung is a traditional material for soil reinforcement in certain area of the world. As a binder and waterproofer, it has the advantage of being cheap and widely available, although it may have pressing alternative uses as fertilisers

or fuel. Bovine blood is used in certain areas as a rendering for adobe. It limits the swelling of clay and confers volume stability on wetted surfaces. Soft cheese is known to have been used in conjuction with lime in mortar, possibly to obtain similar effect as that of Bovine blood but this is with doubtful economic rationality. Also cashew nut residues with high pherolics have a stabilising effect when mixed with soils. Other materials used as local stabilisers for lateritic soil include the fibre reinforcing effects of rice husks, bamboo straw, sugar cane waste, by-products of paper manufacture (lignin and lignosulphonates) and spent sulphate liquor.

It is of interest to at least mention that it has been reported that a complex polysaccharide which is basically a non-ionic cellulose polymer is used by some species of ant to build their hills in Northern Australia. It could be possible that cellulose-based materials may prove to be one of the prospects for further research as soil stabilisers.

2.3.0. DEFORMATIONS

Strick definition of the various type of deformations and associated phenomena in a material will not be easily put in a sentence. However, it can simply be put that deformation is that distortion that a material undergoes when subjected to a stress. There is also deformation as a result of volumetric changes due to hardening and variation in the moisture contents as well as to temperature variation. This distortion is mainly strain distortion. The study of the relation between stresses and strain in most general sense is termed Rheology.

When the deformation increases with time under a constant stress, it is called creep. Creep is not quite synonymous with flow as Orowan / 295 / has pointed out. Flow is said to occur when the law governing the deformation is a functional relation between the rate of strain and stress. Example is the Newton's law of viscosity. In creep, there is an assumption that a relation between stress and strain exist in the first instance, such as Hooke's law or the plastic stress-strain curve of a ductile material, but in addition there is a strain whose presence and magnitude are a function of the passage of time during which the constant

applied stress acts. Thus the stress-strain relation is a function of time, rather than a stress. When the deformation (strain) is kept constant and th-ere is **reduction** in stress with time, it is termed Relaxation. Usually the stress decreases with time. Relaxation and creep are two manifestation of the fact that the relation between stress and strain in a material is a function of time. Relaxation is of considerable importance in the design of pres-tressed concrete structures. However, when a material exhibits changes in strain with time and no external load/stress is acting, due to movement of water from or to the ambient medium, the deformation is termed drying shrinkage or simply shrinkage, and swelling.

Various investigations has been going on in area of Rheology of materials since the early part of the twentieth century. A.M. Neville has investigated into this area for twenty years. Most work in rheology is mainly with concrete materials and lightweight concrete materials. Thus most of the literature will be based on concrete and light weight concrete materials since the composition of the material is close to that of stabilised laterite.

It is not certain who was the first to report that concrete does not in fact behave elastically, however, Neville / 286 / reported that H. I. Woolson described the ability of concrete in a steel tube to "flow" under a high axial stress. As far as behaviour under sustained load is concerned it was probably Hatt /188, 286/ of Purdue University in U.S.A. that first published data on creep of reinforced concrete. He tested a beam of section of 200mm by 250mm. The mix of concrete was the usual 1:2:4 and loaded at third points over a span varying from 2.4 metres to 2.6 metres. The percentage of longitudinal reinforcement varied between 0.75 and 1.50. The beams were loaded at an age of two months in an outdoor location. The result is shown in A—2. Hatt made no reference to shrinkage of concrete, so that the increase in deflection of his concrete beam certainly includes shrinkage effect as well as creep. However, this does not remove the fact that his results are important since they demonstrate the presence of large non-elastic deformation under sustained load. The instantenteous deflection approximately double after two months of sustaining the same load. After analysing his results, Hatt made

a conclusion that the results taken together show a sort of plasticity in concrete, by which it yields under the action of a load applied for a long time, or applied a number of times. Shrinkage which is also a non-elastic deformation was observed even earlier but its structural significance was only recognised when white / 399 / published his paper. However, it was McMillan / 286 /, that distinguished time-dependent deformation of loaded and non-loaded concrete.

Various other investigators had in history observed deformations relating to creep and shrinkage in the early part of the century. Such investigators include Davis and Davis / 101, 102 / Smith /286/ and so on. Thus the problem of the relation of creep to shrinkage, creep and shrinkage has been with us ever since and it should be of interest both from theoretical point of view and for structural design purposes. In addition to observing deformation with time under constant load, Smith / 286 / showed that on removal of load, concrete undergoes both elastic and creep recovery. Thus the broad format of deformation of concrete subjected to sustained load and subsequent unloaded, was established.

2.3.1 DEFINITION OF TERMS

In most general form, the strain-time curve for a material subject to creep is in the form shown in A3. The strain at zero time is primarily elastic although may include a non-elastic components. After that, there are three stages of creep. In primary creep range, the rate of creep decreases with time. When the material exhibits a minimum creep rate, the secondary creep range (sometimes called stationary creep) designates the range of steady state creep. The straight line relation of the secondary creep can easily be noticed if the magnitude of this creep is large compared with primary creep. The tertiary creep may or may not exist, depending on whether or not there is an increase in stress. In concrete, the tertiary creep may arise as a result of increase in microcraking at high stresses.

Illston / 203 / carried out some tests which clearly show the real nature of the magnitudes of primary and secondary creep strain in concrete. In his conclusion, he divided creep into delayed elastic deformation and non-recoverable flow. It is assumed by most researchers that the delayed elastic deformation is the primary creep and what he termed non-recoverable flow as the secondary creep. The results are as shown in A4.

His results clearly show that the primary creep forms a high proportion of creep in the period immediately after application of the sustained load and reaches a constant value after ten weeks. Other researchers have shown slight decrease in the magnitude of the primary creep after the ten weeks' time. However, Neville / 28b / believed strongly that the final magnitude of the primary creep varies little with age at loading or with the previous stress history. Illston carried out his creep test on a concrete mix of 1:6:5 with a water cement ratio of 0.53. While the primary creep varies little with age at loading, the secondary creep increases with time under load but at a continuously decreasing rate / 286 /. Analysis of the test results of Illston by Neville reveal that there is very little or no instantaneous plastic deformation (permanent set). It was suggested that the plastic deformation be ignored in considering strain at loading. However, the rate of both the primary and secondary creep decrease with the age at application of load, indicating that the two phenomena are related to one another. This relation is independent of age. Test results have shown that the primary creep is the same for a given secondary creep for ages at loading between seven days and almost one one year / 28b /.

Neville /286/ considered and investigated also the behaviour of concrete subject to constant load. It is obvious and logic enough for his assumption that the elastic strain decreases with time because the modulus of elasticity of concrete increases with time. Most people who have researched into creep of concrete either believe that creep is the strain in excess of the elastic strain at the time considered or is a strain in excess of the elastic strain at the time of application of load. This however, generated a lot of controversy among researchers until Neville after his various tests came up with what he termed definition of terms. He came up with figures showing the various type of creep occuring when specimens are loaded or un-loaded, in hygral equilibrium with the ambient medium and when the specimen is undergoing drying under load. The figures are shown in A5 for clarity /286/. Thus the environmental condition is an important factor in measuring creep. The actual creep as measured depends on whether the ambient medium is in hygral equilibrium or not, whether or not the sample is loaded or is undergoing drying while at load. This clarify whether or not the creep should be in excess of elastic strain at the time considered or excess of elastic strain at the time of application of load.

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So far it is implied that a sustained stress once applied, continues to act. This can only be so with the selfweight of the member. However, many loads, although sustain do not act indefinitely. When the stress is removed, concrete undergoes an instantaneous creep recovery. Usually, the instantaneous recovery is less than the elastic strain at the time of application of load. This instantaneous recovery is followed by a time-depended recovery known as creep recovery shown in A6.

The magnitudes of the creep and the creep recovery are such that the creep recovery is less than the creep except in exceptional cases where the concrete is subjected to the sustained stress at a very advance age. However, even at this time, the maximum recovery can only be equal to the creep and never greater. Thus the concrete sample subjected to a sustained stress and subsequently unloaded exhibit an irrecoverable or residual strain sometimes referred to as permanent set.

The value of the instantaneous strain at the time of application of laod depends on the measuring technique used in a particular case. If load is applied slowly, some creep may take place. The

solution of very rapid loading method is often very difficult to achieve. However, the elastic strain may be determined from measurement of the change in strain under a fluctuating stress of small magnitude. It has been observed that the elastic strain calculated from the dynamic modulus of elasticity has probably the most correct value from the theoretical standpoint but is not convenient to use. For most practical purposes, the assumption that the elastic strain on loading is not distinguishable from the observed instantaneous deformation is convenient. This assumption makes it possible to separate out the two important types of deformation: that which occurs on application of load and that which occurs with the passage of time- while the load continues to act. It is worth mentioning however, that no definition of creep is inherently more acurate than any other since we cannot isolate on the basis of our understanding of the phenomena involved. There could be creep from other strain occuring during the period the load is acting. However, physically incorrect definition would hinder the study of the phenomena involved as the understanding of creep mechanism seems to be increasing.

2.3.2 TYPES OF DEFORMATION

The phenomenon of creep in concrete has been variously termed flow, plastic flow, plastic yield, plasiic deformation, time yield and time deformation /286/. This might have arisen as a result of the concept of the mechanism of deformation as seen at the time and partly from a lack of agreement on what was still a newly discovered phenomenon. Nowadays it is widely accepted as creep.

To understand the various types of deformations, it is worthwhile to know the nature of the materials especially concrete that undergoes these deformation Although under many circumstances cracks are visible in concrete members and sometimes even crushing may be observed, concrete is not a brittle materials. For a material to be brittle, such material is considered to develop only a small strain before fracture. In concrete, large strains are possible prior to failure. Example is reinforced concrete column which usually fail in a ductile manner while reinforced concrete beams can develop large rotations, leading to a considerable redistribution of stress and a comparable ductile failure. This behaviour of concrete has contributed to its success as a structural material.

In composition, concrete is a multi-phase composite material, consisting of particles of coarse aggregate embedded in a matrix of mortar. Mortar itself consists of particles of fine aggregate embedded in a matrix of cement paste. At the microscopic level, the cement paste consists of grains of unhydrated cement embedded in a matrix of products of hydration of cement. These products are cement gel with a semi-continuous system of water-filled or empty capillary pores. At the submicroscopic level, the cement gel is a mixture of ill-formed intertwined particles with some fibrous or needle-shaped, but mostly sheets and foils which form a continuous matrix with a continuous system of water-filled gel pores.

Hydrated cement paste plays the dominant role in time-dependent deformation of concrete and in fact is said to be the seat of creep of concrete. The aggregate only modify the behaviour in a quantitatively important way. The chemical reaction between cement and water leads to the formation of hydration product of colloidal size and properties of about one hundred Angstroms in cross-section. The resulting mass of cement gel is about twenty eight percent by volume and very porous with pores of diameter of about fifteen Angstroms /286/. It is estimated that there are about 7×10^{10} gel

particles per cubic metre /286/ and because of their size, the shape of the individual gel particles cannot be determined by observation. There are, however, strong indications that they are in the form of thin, rolled or crumpled sheets, averaging ten thousand Angstrom in length, about twenty to thirty Angstrom thick, and one hundred Angstrom wide in the rolled state. Each sheet is composed of two to three solid monomolecular layers with a monomolecular layer of zeolitic (interlayer) water in between. Thus there is water between the layers of a sheet, within convolutions of a sheet, between the particles, and the capillary water between agglomerations of particles. Due to small distance betweer adjacent solid surfaces, they are subject to mutual forces of attraction, which is quite considerable in magnitude. The force-field is affected by the presence of the absorbed water. The forces of solid-to-solid attraction are balanced by compressive stresses within the solid. It is possible that there are, in fact, points of contact between the individual solid particles through which compression is transmitted when the cement paste is not saturated but the solid bearing area is only a small fraction of the total cross-section. However, the significant thing is that while macroscopically, hydrated cement paste is a continuous porous mass with wide spectrum of voids,

microscopically, the paste is an aggregation of anisotropic particles of colloidal dimensions, interspersed with a crystalline phase.

Rheology recognises three fundamental types of deformation. These include elastic, plastic and viscous. These deformation can also appear in combinations, such as elasto-plastic or visco-elastic and the combination can be complicated further by factors such as aging, thixotropy or strain hardening.

Elastic deformation can be defined as an instantaneous, linear and completely reversible deformation of crystals or molecules. Delayed elasticity can be considered as a form of creep, usually characteristic of a disorderly molecular arrangement. The deformation occurs at a decreasing rate and is fully reversible because the energy producing it is not dissipated but stored in the material. If a body consisting of a solid and a liquid phase is subjected to a sustained stress, the instantaneous elastic deformation is governed by the mean modulus of elasticity of the combine system. Thereafter, the liquid flows and the stress on the solid phase increases with a resulting increase in deformation.

Viscous flow applies to ideal fluids and requires that the rate of strain with respect to time be proportional to the applied stress. Plastic deformation on the other hand is due to wholly irreversible changes of position of single units making up a solid body, that is in a simple terms: distortions without a volumetric changes. This is only for a stress greater than a limit value. Since the changes are irreversible, the strain does not change when the stress is removed. Plasticity is often connected with crystalline order of molecular arrangement and may be associated with crystallographic slip but can also occur when there is a considerable degree of atomic fit between neighbouring molecules /286/. A Purely plastic material would not, strictly speaking show creep as the application of a stress would produce a corresponding strain.

As for the behaviour of concrete under a sustained load, within the normal range of working stresses, Hansen /175/ classified the deformation he observed into recoverable and irrecoverable ones as shown below.

Deformation	Instantaneous	Time-dependent
recoverable	Elastic	Delayed Elastic
Irrecoverable	Plastic (Permanent set)	Viscous

Both deformation of the time-depended column represent creep however, Hansen considered the delayed elastic deformation to be the primary creep and the viscous deformation (ie the residual deformation to be the secondary creep). Neville /286/ in analysing the results believed that the definition of total creep as the sum of load-induced time-dependent deformations is acceptable, however, the allocation of recoverable and irrecoverable deformations to delayed elastic and viscous flow is highly debatable. This is because there is some doubt whether the irrecoverable part is really viscous or possibly plastic. These two types of deformations are clearly defined physically and rheologically as earlier defined. Viscosity is that behaviour of a material when the potential energy of an external force applied to the material is relatively small compared with the thermal energy of the molecular structure. However, if the energy of the applied force is is larger than the thermal energy, the deformation

is considered to be plastic. When the irrecoverable deformation increases with the duration of action of the load, the material is said to flow. However, the plastic flow is characterised by a value of stress called plastic limit, below which there is no flow. The absence of plastic limit characterises the fluid flow.

In concrete, it has been firmly established that there is no plastic limit so that rheologically, concrete is a fluid. Thus, the irrecoverable time-dependent deformation should be designated as viscous. This explanation is further supported by the approximate proportionality between stress and the rate of strain at normal working stresses. These working stresses are those below pronounced microcracking. Hansen /182/ further supported his argument in favour of viscous flow by saying that the principle of superposition of strains approximately holds good for concrete and this principle is valid only in elastic or visco-elastic materials, since no superposition can be applied to plastic deformation.

However, looking at the above reasons carefully and if a coefficient of viscosity is calculated for concrete, it varies not only with the age of the concrete but also with time under load at the early

period of application of the load. This shows that the time-dependent irrecoverable deformation is not purely viscous but includes some permanent set due to closing up of the gel structure. This may be the probable reason why the principle of superposition is believed by some investigators to overestimate the creep recovery. It is also worth mentioning that Nielsen /286/ carried out a test which have shown that the magnitude of the linear visco-elastic component of strain in concrete is independent of the time at which load is applied if the application of load takes place at the time the degree of hydration has reached sixty five per cent (about 28 days strength)

There are strong arguments against the viscous flow concept by Neville /286/ and Illston /203/. They argued that in viscous flow, the rate of deformation at any instant is independent of any previous deformation. Thus deformation during a given period of time under load starting at age T_1 should be the same as the increase in deformation Γ^{at} age T_1; when the same load had been acting since an earlier instant. Illston data from tests with different ages at loading showed that the above requirement is not fulfilled by concrete. His data is presented in A7. Another result of the influence of past deformation in concrete was indicated by test carried out by Vogt /390/. His results show that if load on a

specimen of concrete is increased after a period under sustained stress, the increase in strain is small until the stress-strain curve approaches the virgin curve which is obtained by a direct increase in stress from zero to the higher value.

In contrary to the viscous deformation, plastic deformation is influenced by previous deformation, provided the past stress exceeds the plastic limit. This is why concrete is believed to exhibits plastic behaviour with a plastic limit of zero. Nevertheless, the deformation of concrete is basically different from the plasticity of, say, steel as evidenced by the fact that no redistribution of moments takes place in hyper-static concrete structures under normal working loads. From the gathered literature, it seems that the classification of deformation or types of deformations in concrete is somewhat obscured. Deviations from linear stress-strain or stress-rate of strain relations are most likely due to micro-cracking at the aggregate-matrix interface. This is of course termed fracture and not flow, but, since the rheological nomenclature is based on phenomenological behaviour of materials, the apparent yield caused by micro-cracking can be designated as plastic flow.

For practical purposes of structural design, the situation of deformation in concrete can be assumed as follows. The behaviour of concrete as far as stresses due to external loads are concerned, is sufficiently close to visco-elasticity. Sustained loads produce no redistribution of stresses or reactions both in plain and in reinforced concrete /286/.

2.3.3. CLASSIFICATION OF DEFORMATION

As earlier stated, an attempt to classify the deformation occurring in concrete may be difficult, however the classification is based on the already observed deformations. Deformations occurring in specimens of concrete can be classified into three basic deformation. These include shrinkage, creep and relaxation deformations. The stress sustained by the sample specimen will determine whether the deformation is of elastic type, viscous or visco-elastic types deformation. As defined earlier, creep is a deformation occurring under, and induced by, a constant sustained load while relaxation is a decrease in stress with time under a constant deformation. The changes in strain with time when no external stress is acting due to movement of water from or to the ambient medium is termed shrinkage.

2.4.0. SHORT AND LONG TERM DEFORMATION

Several attempts have been made to relate the ultimate creep (creep at infinity) of concrete, to the elastic deformation on loading, but the ratio of the former to the later varies widely, probably between 2 and 5. For a given mix however, the ratio of ultimate creep to the elastic deformation on loading depends on the storage conditions, and the German Building code for prestressed concrete DIN4227 /116/ recommend a set of values of the ratio ($\Phi_\infty = \frac{C_\infty}{C_i}$) for different conditions of storage. The values as published in 1953 are as given in A8. Bureau of Reclamation after their investigation /383/ appears to say that creep can be estimated from the change with time in the elastic properties of concrete. Appendix A - 9 shows a graph of specific creep at any time plotted against the ratio of modulus of elasticity at that time to the modulus at the time of application of the load. The data cover a wide range of ages at which the load was applied. While such a relation is a useful parameter in estimating creep for a given mix of concrete; and the modulus is easier to determine than creep, the necessary information can be obtained only after a long period. Furthermore, it is not considered fundamentally correct that the

modulus of elasticity of concrete is the only parameter in the magnitude of creep /286/.

The magnitudes of shrinkage and creep are of the same order as the elastic strain under the usual range of stresses, so that the various types of strain must be at all times be taken into account. The increase in strain while the load, or part of it, is acting is due to creep, but the dependence of instantaneous strain on the speed of loading makes the demarcation between elastic and creep strains difficult. For practical purpose, an arbitrary distinction is usually made; the deformation occurring during loading is considered elastic and the subsequent increase in strain is regarded as creep. The modulus of elastic safisfying this requirment is the secant modulus, (Appendix A 10). There is no specific standard method of determining the secant modulus; in some laboratories, it is measured at stresses ranging from 3 to 14 MPa while in others at of stresses representing 15, 25, 33 or 50 per cent of the ultimate. The secant modulus decreases with an increase in stress, as such, the stress at which the modulus has been determined should always be stated. The secant modulus is a static modulus since it is determined from an experimental stress-strain relation, in contradiction to the dynamic modulus.

2.4.1 FACTORS AFFECTING SHORT-TERM DEFORMATION

In considering the deformational properties of any material under load, a distinction is usually made between the deformation due to external loading; which is usually elastic deformation and creep, and the volumetric changes due to hardening and variation in the moisture contents as well as to temperature variations which in this case is termed shrinkage. In this section, consideration is given to the deformation occurring as a result of external load of short-duration - (elastic deformation). It has been observed that repeated loading and unloading reduces the subsequent long-term (creep) deformation that may occur in the short duration loading, so that the stress-strain curve on third or fourth loading exhibits only a small curvature /281, 319/. Thus in the German Code DIN 1048 paragraph one, and as further explained in the lecture notes /116, 319 /, the material under short-term load duration for modulus of elasticity test is loaded and unloaded ten times within a range of stress of 2 and 33 per cent of the ultimate strength before reading the strains for the E-modulus. This completely eliminate the creep strains. The graph explaining the repeated

loading process is as shown in A-11. Thus one can practically say that the E-modulus obtained in this manner is purely without any additional creep strains. Hence the short-term deformations is referred to in most part of the thesis as the Elastic modulus of concrete or the material ~ concerned. The modulus of elasticity of a material is however depends on the various other parameters of the materials. The condition of the specimen for the E-modulus test is a factor which affect the value; a wet specimen has a higher modulus of elasticity than a dry one, while high strength concrete has higher modulus than lower strength concrete. The properties of aggregate also influence the m ulus of elasticity, though they do not generally affect the compressive strength. The higher the modulus of elasticity of the aggregate, the higher the modulus of the resulting concrete or composite material. The shape of the coarse aggregate particles and their surface characteristic also influecne the value of the modulus of elasticity and the curvature of the stress-strain curve of the composite material /25, 56, 78 183, 281 393/. The relation between modulus of elasticity and strength depends also on the mix proportions; since aggregate generally has a higher modulus than cement paste; and on

the age of the specimen. At later ages the modulus increases more rapidly than strength. This is shown in graph A 12. The modulus of light weight aggregate concrete is usually between 40 and 80 per cent of the modulus of ordinary concrete of the the same strength. Since the modulus of lightweight aggregate differs little from the modulus of the cement paste, it thus appears that mix proportions do not affect the modulus of elasticity of lightweight aggregate concrete /80, 281, 297/. The relation between the modulus of elasticity and strength is unaffected by temperature of storage up to about 230°C for concrete for possibly higher, since both properties vary with temperature in an approximately the same manner. However, for the same strength, the modulus is somewhat higher the lower the early curing temperature; thus steam-cured concrete has a lower modulus than moist-cured concrete of the same strength but the difference is under 10 per cent. However, results of investigation by Haque and cook / 185 / show that the dynamic modulus of elasticity decreases on drying and increases on resorption. The process is however not reversible as on resorption there is only a partial regain of the modulus. Thus the first drying induces some irreversible changes in the structure of the

hardened cement paste. The preposition that shrinkage induces microcracking is a possible mechanism responsible for the reduction of the modulus on drying. Research work has shown /2/ that the content of the oxide of sulphur (SO_3) in cement has some effect on long-term deformations. However, this has little or no effect on the modulus of elasticity of concrete especially. /9/. In general, the modulus of deformation of concrete material or composite material depends on the moduli of deformation of its components, the compressive strength of the concrete or composite material, its density, pores, bond between the cement paste and aggregates. /142, 281, 286./.

2.4.2 EXPERIMENTAL RELATIONS ON SHORT-TERM DEFORMATION

The stress-strain diagram of concrete or material concerned indicates a number of distinct stages in the stage of the material, thus a single formular to encompass the stages always lead to small deviation at the initial point of the curve. However most mathematical relations available seems to be the single expression to relate the different stages. Blechman/66/ seers to be proposing two expressions but is yet to be tested and accepted. However, it is worth noting that the continous flow of papers in this area has lead to additions of further formulae (models) for the stress-strain curve description. Thus the expressions given below are those which seems related to this thesis.

The summary of the varicus expression are as given below

TABLE 1 MATHMATICAL EXPRESSIONS FOR SHORT TERM DEFORMATION

Equation Number	Expression	Author	Ref
1	$E_c = 9.1 \left[f_{cu}\right]^{0.33}$	CP110 1972	/281/
2	$E_c = 1.7\ \rho^2 \left[f_{cu}\right]^{0.33} \times 10^{-6}$	CP110 1972	/281/
3	$E_c = 1.6\ \rho^2 \left[f_{cyl,k+8}\right]^{0.33} \times 10^{-6}$	CEB	/281/
4	$E_c = 4.73 \sqrt{f_{cyl}'}$	ACI	/281/
5	$E_c = 43\ \rho^{1.5} \sqrt{f_{cyl}'} \times 10^{-6}$	ACI	/281/
6	$\sigma = \frac{E\varepsilon}{1-\left(\frac{\varepsilon}{\varepsilon_{max}}\right)^2}$	Desayi & Krishnan	/281/
7	$\frac{\sigma}{\sigma_{max}} = \frac{\beta\left[\frac{\varepsilon}{\varepsilon_{max}}\right]}{\beta-1+\left[\frac{\varepsilon}{\varepsilon_{max}}\right]^{\beta}}$	Carreira & CHU	/79/
8	$\frac{\sigma}{\sigma_{max}} = \frac{\varepsilon}{\varepsilon_{max}} \exp\left[1 - \frac{\varepsilon}{\varepsilon_{max}}\right]$	Baldwin & North	/28/
9	$\frac{\sigma}{\sigma_{max}} = \frac{A\left(\frac{\varepsilon}{\varepsilon_{max}}\right) + B\left(\frac{\varepsilon}{\varepsilon_{max}}\right)^2}{1 + C\left(\frac{\varepsilon}{\varepsilon_{max}}\right) + D\left(\frac{\varepsilon}{\varepsilon_{max}}\right)^2}$	Fanella & Naaman	/133/
10	$\sigma = \frac{A}{1 + B\varepsilon + C\varepsilon^2 - D\varepsilon^3}$	Desayi, Iyengar & Reddy	/115/

Equation Number	Expression	Author	Reform
11	$\frac{\sigma}{\sigma_{max}} = \frac{\epsilon}{\epsilon_{max}} \left[2 - \frac{\epsilon}{\epsilon_{max}} \right]$	Olesz	/292/
12	$\epsilon_{max} = \epsilon_u + \frac{1}{E_u} \sigma_{max}$	Olesz	/292/

Equation 1 /281/

In this emperical expression, the terms has the following meaning:-

E_c = modulus of elasticity in GPa.

f_{cu} = cube strength of concrete N/mm^2.

Equation 2 /282/

E_c = modulus of elasticity in GPa

ρ = density of concrete or material kg/m^3

f_{cu} = cube strength N/mm^2

Equation 3 /281/

This expression which is also emperical is given by CEB for lightweight aggregate

E_c = modulus of elasticity N/mm^2

ρ = density of concrete or composite material

$f_{cy}l$ = specific characterisitic cylinder strength N/mm^2.

The additional factor 8 is introduced to this expression in an attempt to compensate for the difference between the specific characteristic strength and the actual mean strength of the concrete produced.

Equation 4 /281/

E_c = modulus of elasticity N/mm^2

$f_{cy}1$ = specific characteristic cylinder strength N/mm^2

and this expression is for normal weight concrete. When the density of the concrete is different, the ACI proposes the expression in equation 5.

Equation 5 /281/

E_c = modulus of elasticity

ρ = density

$f_{cy}1$ = characteristic cylinder strength.

This expression differs from the European concrete Board approach in that the influence of density is taken as being to the power 1.5 instead of 2.

All the above emperical relations are valid in general terms and are affected by the condition of the specimen at test. The properties of aggregates also affect or influence the value of the modulus obtained using the above expression.

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Equation 6 /281/

In this stress-strain curve characteristic expression

σ = stress N/mm^2

E = initial tangent modulus, assumed to be twice the secant modulus at maximum stress N/mm^2

ε = Strain

ε_{max} = Strain at maximum stress or maximum strain.

This expression is said to show a considerable promise both in the ascending and descending part of the curve.

Equation 7 /79/

This expression is a general form of the serpentine curve which is modified to represent the complete stress-strain relationship for unconfined concrete.

This expression as tested is said to fit a wide range of testing conditions of concrete for both the ascending and descending branches of the stress-strain curve in compression. The terms in the equation are explained below.

σ = stress N/mm^2

σ_{max} = maximum stress

ε = strain

ε_{max} = maximum strain

β = is a material parameter that depends on the shape of the stress-strain curve.

Equation 8 /28/

This expression is communicated personally to the authors by one P. H. Thomas. This relation represent the effect of temperature upon the relationship between stress and strain for contrete under compression up to

700^{o}c. It is derived from the scaling law and it expresses the variation of the modulus of elasticity with temperture. The terms are the same meaning as is in equation 7 above.

Over the whole range, $\frac{\varepsilon}{\varepsilon_{max}} > 0$ .In the early part of the curve, for small values of $\frac{\varepsilon}{\varepsilon_{max}}$, the curve is approximately linear, as such the authors expressed that early part by an expression gives below.

$$\frac{\sigma}{\sigma_{max}} = K \frac{\varepsilon}{\varepsilon_{max}} \qquad (8(b))$$

and the value of K is given to be equal to 2.18 and can be determined at room temperature.

Equation 9 /133/

In this expression, an analytical relationship is proposed by the authors to predict the complete stress-strain curve of fibre reinforced mortar in compression. According to Fanella and Naaman /133/ it can be used as a first approximation in design. The terms in the equation means the following:-

stress ratio $\frac{\sigma}{\sigma_{max}}$ refferred to as normalized stress

strain ratio $\frac{\varepsilon}{\varepsilon_{max}}$ refferred to as normalized strain.

σ, σ_{max}, ε, ε_{max} retain their normal meaning.

A, B, C, D are constants to be determined from the boundary conditions of the curve in question.

Equation 10 /114, 115/

This proposed expression characterises the stress-strain curves of concrete confined in a circular steel spirals subjected to monotonic compression. The terms σ and ϵ retain their usual meaning. This expression represent satisfactorily both the ascending and descending branches of the stress-strain curve in compression for unconfined as well as confined concrete. The parameters A, B, C, D are to be obtained from the boundary conditions and test results. They are constants. This equation was earlier suggested by Saenz /281/ for unconfined plain concrete. From the work of Desayi et al /115/ this expression is found convenient to use for confined concretes as well. Comparing equation 10 to that of 9, one observe that the expressions are almost the same, while equation 9 are in a ratio form, that of 10 is not and the conditions for determining the constants are both from the boundary condition of the curve in question.

Equation 11 /80, 292/

This expression as proposed by Oleszkiewicz gives the characteristic of the lightweight aggregate concrete deformation under external load of short duration. In other words, it gives the stress-strain curve characteristic for lightweight aggregate concrete.

The terms in the equation is the same as earlier explained.

According to Oleszkiewicz, any concrete or composite material has a characteristic deformation property which is as a result of the sort of aggregate and matrix used. Thus, from the physical point of view, any concrete even the weakest one, as a brittle material, has a clear deformation limit and after its excess the resistance of concrete against the external loading rapidly decreases. On approaching the smallest strength of a given type of concrete, theoretically zero, its lowest ultimate strain may be established. To determine the increase of the ultimate strain due to increase of the concrete strength, a concept of a modulus of ulimate deformation increament was introduced. This led to equation 12.

Equation 12 /292/

The terms of the equation is given below

ε_u = is the lowest ultimate strain for a given type of concrete as its crushing strength approaches zero.

E_u = is the constant modulus of deformation increament for a given type of concrete N/mm^2

The rest terms remains as before.

2.4. 3 FACTORS AFFECTING LONG-TERM DEFORMATION

Creep of materials is often termed long-term deformation since the deformation is a function of time. Creep of concrete is sensitive to almost any property of the hardened concrete, as such, the factors affecting or influencing creep is a very wide topic. Some of these influences are minor, many are indirect and can be accounted for by other properties of concrete. However, some of the factors are mentioned and ways on which they influence the creep of concrete are explained. The factors discussed in this thesis include the influence of cement and admixtures, aggregates, strength, stress, humidity and temperature.

(a) Influence of Cement and Admixtures

This factor is mainly concerned with the properties of the ingredients of the concrete mix. It cannot therefore be discussed under one sub-heading as such the influence of cement and admixture is further sub-divided into the following sub-heading for easy understanding and clarity.

Type and Composition of Cement

Logically, cement is the most important factor in long-term deformation (creep). This is because the hydrated cement paste is the seat of the phenomenon of creep. The influence of cement is two-fold: that due to the chemical and physical properties of the cement, and that due to variation in the amount of the hydrated cement paste.

Neville /277/ in his experimental investigation on the creep of mortars made with cements of various chemical composition, originating from different cement plants, thus having different kiln, temperature histories, degree of Carbonation, period of storage, and amount of surface hydration have not shown any direct correlation between creep and the chemical composition of cement. This does not mean that the compound composition of different type of cements has no effect on creep but rather that the chemical composition of a particular brand like portland cement from various factories has no effect on creep. However, despite the assumption made in the Boque equation, giving rise to glass content, there are some broad influences on creep of the cement composition as characterised by the type of cement of the ASTM classification. In general, creep seems to be inversly proportional to the rate

of hardening of the cement used. Thus, for a given age of loading, creep is in increasing order of magnitude for concrete made with the following types of cements: High-alumina, Rapid-hardening Portland (Type III ASTM classification), Ordinary Portland /Type I/ Portland blastfurnace, Low-heat (Type IV) and Portland Pozzolan. At the same age, these cements have achieved varying proportions of their final strength. It thus therefore seems logical that the more hardened - the paste, the more rigid it is and the lower its creep potential at a given applied stress. The comparision of creep of the different cements in the preceeding discussions under this subheading is based on specific creep (creep per unit stress). If however, the stress applied in each case bears the same proportion to the strength at that time, the type of cement per Se does not influence creep, although the rate of hydration while the specimen is under load is a factor and this varies for different cements mentioned above. This is related to strength which is discussed later.

Tests carried out by Glanville and Thomas /160/ have shown that the differences in creep between Type I and Type II cements are smaller

when the specimen are stored in water. This can possibly be explained by a lesser difference in the degree of hydration compared with storage in air.

Typical data / 286 / for creep of concretes made with different cement are as shown in A-13 , A-14 and A-15 . All the concretes used had a water-cement ratio of 0.55 and aggregate-cement ratio of 5.4. The stress-strength ratio was one-third of the cube strength. Properties of the various mixes at the time of application of load is also given in A-16 .

From the above data, it can be seen that type I cement concrete always exhibit the highest creep, being followed by Type III cement and then by high alumina cement. Hummel / 20 / observed that the specific creep was lower for a higher initial strength. This was proved from the observed values of creep after one day of application of load, in which there was little further increase in the difference in creep between the different cements.

The creep behaviour of Portland blastfurnace was investigated by Ross / 328 / and found that the creep-time curves of Portland balstfurnance cement have the same shape as that of Type I cement but the creep was however, larger with the portland blastfurnance. For instance a 1:2:4 mix of portland

blastfurnance cement with a water-cement ratio of 55 per cent and a strength of 25.7 N/mm^2, kept for three years under a stress of 7.2 N/mm^2, showed a specific creep of 3.29×10^{-4} per N/mm^2. Other comparative tests by Ross /328 / of Type I and Portland blastfurnance cement concretes of the same proportions, have shown a markedly greater creep in the later. The situation was found to be the same in the comparison of flexural specimens of neat cement paste.

However, test by Okada et.al / 289 / showed a lower creep when blast furnance slag is used as a partial replacement of cement, especially when the slag is of high content. This was further complicated by the fact that the creep was lower though the strength of the replacement was lower, the applied stress being the same in all cases. No explanation of this behaviour was however available but one believe that the performance shown by Ross /328 / tests is more acceptable and typical for the European and American blastfurnance cement.

Replacement of portland cement by Pozzolana increases creep. This is probably because the strength of the mix at early ages is reduced. Of the various pozzolanic materials, diatomaceous earth is reported to lead to the highest creep. More recent tests by Neville / 286 / and various other

data have shown that at a given initial stress-strength ratio, replacements by fly ash and similar materials influences creep only if the long-term rate in gain of strength of concrete is affected.

Additives And Air Entrainment

Data on creep of concrete with special additives appears to be little compared to the rest factors influencing creep of concrete. Patapov and Zalan / 309 / carried out creep test on concrete made with a furfurol-acetone monomer which had a high furfurol content and a two percent addition of a sodium fluosilicate filler. Such concrete is said to have a high chemical resistance and may be used in an aggressive environment. The strength of a particular mix used was 950 kg/cm^2. Potapov and Zalan observed that at a stress-strength ratios below 50 percent, the creep ceases after two to two and half months. By the time the creep deformation reaches 10 to 50 per cent of the elastic deformation, creep is

no more proportional to the stress-strength ratio even for low values of the ratio and under some conditions, a stress-strength ratio as low as 52 per cent can lead to time failure after a few months.

Satalkin / 344 / carried out investigation on the influence of polymers on creep. His test was carried out on the following type of polymer concrete: (A) - 1:3 portland cement mix without polymer; (B) - portland cement mix with 20 per cent polyvinyl acetate emulsion; (C) - only polymer mortar based on furfurol - acetone monomer. In his result, he observed that the creep exhibited by the polymer mortar (C) is three times that of the portland cement (A) and that of portland cement mixed with 20 per cent of polyvinyl acetate emulsion (B) is about two times that of the portland cement mix (A). They were all loaded at the same time with the same stress-strength ratio for about four months. The results suggest higher creep for polymer concrete.

Gvozdev / 167 / worked on the influence polymer and resin concretes on creep. He observed from his results that polymer and resin concrete exhibit much higher creep than ordinary concrete. This agrees with the findings of Satalkin / 344 /. For expoxy concretes, Gvozdev observed a moderate

increase in the creep, but related to the amount of the expoxy in the mix. After six months under load, the rate of creep of epoxy concrete decreases consd-erably such that at about one year under load, the creep of lean expoxy mixes is approximately the same as that of sealed portland cement concrete specimens under the same stress / 383 /. On the basis of stress-strength ratio however, epoxy concrete exhibits creep of about three times higher than that of mass concrete. Concrete which is impregnated with a monomer solution, with polymerisation accomplished by gamma radiation or by thermal treatment seems to exhibit vitually no creep. Whether this is due to residual stresses or to other causes is not yet known.

The U.S. Army Engineers / 381 / had worked on the influence of air entrainment in concrete on creep. After analysing the laboratory results, it indicates an increase in creep due to air entrainment; for instance after one year under load applied at 28-days, concrete with 5.4 per cent entrained air showed 40 percent more creep than non-air-entrained concrete which had an air content of 1.7 per cent. However, the presence of entrained air affected the mix proportions

so that the mix with entrained air had an aggregate content of about 73 percent, compared with 70 per cent for the non-air-entrained mix. This would at least in part, explain the higher creep of the concrete with entrained air. It should be noted that the changes in mix composition did not affect the creep and elastic strains to the same extent. For concrete loaded at 28 days for eleven months or at ninety days for nine months, creep was equal to the elastic strain when no air entrainment was used, but in the air-entrained mix, creep was smaller than elastic strain.

Jones et.al / 218 / investigated the effects of air entrainment in lightweight aggregate (expanded clay) concrete. They concluded that the net effect of air entrainment is usually to increase creep. However, their air content is of below 5 to 6 per cent which is the normal practical range.

Tests by Ward et.al /394/ on concrete made with normal weight aggregate have shown that the creep of mixes having the same workability and strength at loading is not affected by the presence or absence of entrained air. Thus the relation between creep and stress-strength ratio applies

to air-entrained concrete as well as to plain concrete.

It appears thus that under practical conditions, air entrainment is not an important factor in creep. Air bubbles represent aggregates with a zero modulus of elasticity so that the resistance to creep deformation is less than when there are no voids present. On the other hand, air entrainment generally improves the workability and, therefore allows the use of a lower water-cement ratio of a leaner mix. The latter leads directly to lower creep while the former has a similar effect through an effective reduction in stress-strength ratio (for a given applied stress). Thus the net effect of air entrainment in concrete on creep is probably generally not significant.

(b) INFLUENCE OF AGGREGATES ON CREEP

In concrete, the cement paste is interspersed by a second phase, aggregate, which does not undergo creep. This does not mean, however, that aggregate does not influence creep. In fact, the modification of creep by aggregate can be expressed in terms of its quantity and its mechanical properties.

INFLUENCE OF AGGREGATE CONTENT

The aggregate content and the content of the hydrated cement paste, that is their volumes as a fraction of the volume of hardened concrete are complementary. Thus, considering one or the other is strickly equivalent. However, studies of the aggregate content and that of the related richness of the mix have been made, but because the influence of water/cement ratio on creep was not known when some of the earlier tests were made, conflicting conclusions were reported. Glanville /159/ found creep to increase with decrease in the cement paste content. All the mixes compared had the same slump, so that with an increase in the aggregate cement ratio, the water/cement ratio was also increased. It was thus the latter factor (water/cement ratio) that had a dominating influence on creep.

Arnstein and Reiner /21/ found that increasing the aggregate content reduced the rate of creep of

mortars but increased that of concretes. L'Hermite /234/ observed that creep decreases with an increase in the amount of cement paste but this is offset by an increase in shrinkage; the creep of rich mortars is, however, increased. Neither Arnstein and Reiner's nor L'Hermite's statement has been confirmed from tests.

Lorman /240/ used contour plotting in an attempt to assess the effects on creep of the quantities of water, cement and aggregate in the mix, and found creep to be practically independent of the cement content. If, however, instead of the quantity of cement per dry mix used, the cement paste content on volumetric bacis is considered, the tests are found to be only within a narrow range of paste contents of 28 to 36 per cent. Within this range, out of 6 mixes tested were lying between 31 and 33 percent. It is thus not suprising that the quantity of cement was not found to be significant factor in creep- as reported by him.

Jones et al /218/ in their conclusion stated that adding cement to a lightweight aggregate concrete at constant water content decreases creep. However, little did they realise that increasing the cement content of the mix at a constant water content leads not only to an increased cement paste content but also higher strength of the mix and decrease in water cement ratio.

The later effect decreases creep, and may offset or even exceed the increase in creep due to the increased cement paste content.

Ishai /206/ on the other hand reported that the magnitude of creep increases with an increase in sand concentration. However, his mixes with high sand content contained a large volume of voids, arising from a lack of cement paste to fill the void present. These voids affect adversely the strength of the mortar and hence increased creep. Thus the sand concentration, as measured by Ishai, is not a real factor in creep.

In tests on mortar specimen of about 2mm thick, de La Pena /112/ found the following relative magnitude of creep for different mixes with water/cement ratio adjusted to suit work bility.

Sand-cement ratio	0.25	0.67	1.50
Percentage of cement by weight	80	60	40
Relative creep	1.00	0.74	0.49

This pattern indicates that the higher the cement paste content the higher the creep, and more specifically, that replacing one part of cement by one part of sand (by weight) decreases creep in a direct proportion if expressed in terms of creep of the richest mix.
This conclusion is opposite that of Glanville /159/ and Jones et al /218/. Even though the specimens seems to be so small and the tests were just a set of results, later tests carried out by other investigators seem to

agree with de La Pena's conclusion. He also further found that the ratio of creep deformations of a given lean mix to a given rich mix is higher the lower the applied stress-strength ratio. In other words, the influence of richness increases with an increase in stress-strength ratio.

Polivka et al /308/ measured creep of what he called screened concrete with a large maximum size aggregate and suggested creep is proportional to the cement paste content. By screened concrete, he meant that sizes of aggregate too large to be included in the specimen are removed by wet sieving. This approximation of creep proportionality may be convenient only under a given set of conditions. Thus the "rule" is unlikely to satisfy the physical conditions in the concrete. If creep were directly proportional to the cement paste content, then it could be believed that the aggregate floats in a matrix of cement paste and that the stress in the aggregate does not change significantly with creep. The unrestrained "floating" concept does not seem to conform to reality, especially in mixes with a large maximum size of aggregate where the cement paste may represent as little as 12 to 20 per cent of the total volume of concrete.

The reason why there have been conflicting data and at the same time no definite influence of the cement paste content on creep earlier was due to the

fact that majority of practical mixes have range of values of the paste content very small. With the exception of special uses, such as mass concrete or factory-made concrete, concrete is usually with a workability such that lean mixes require higher water/cement ratios than rich mixes, and the increase in water/cement ratio represents an increase in the volume of the cement paste. Thus comparing three mixes with aggregate/cement ratio of 9, 6, 4.5, and corresponding water/cement ratio of 75, 55 and 40 per cent, one observe that the cement paste content to be 24, 27 and 29 per cent respectively which is a very narrow range.

(c). INFLUENCE OF STRENGTH, AND STRESS

One admits that strength is affected by the properties of the constituents of concrete but it is also a function of the quality of cement paste and age. The way creep response to stress depends on the relationship between stress and strength, thus it is more convenient to include the relation betweeen stress, strength, and creep in the same section. However, one is still concerned with the influence of the intrinsic factors on creep but the emphasis is moving to a derived mechanical property which is the strength.

STRESS - STRENGTH RATIO

The investigation by Neville / 275 / on the influence of the type of cement on creep led to the discovery of an approximate stress-strength ratio rule. This rule states that for a constant mix proportions and the same type of aggregate,

creep is proportional to the applied stress and inversely proportional to the strength at the time of application of the load. There are limitations to this rule but then, it is more convenient to consider the influence on creep of the level of stress, other conditions including the strength being constant.

From Neville's wide range of experimental result in 1959 / 275 / there seems to exist substantial evidence of a linear relationship between creep and the applied stress except in specimens loaded at an early age which is one to three days. However, there is doubt as to the upper limit of this relation. The lower limit may be virtually be at zero stress as creep is exhibited by concrete even at low stresses. In terms of stress-strength ratio, an upper limit between 0.30 and 0.75 has been observed. Freudenthal and Roll in 1958 /137 / reported a stress-strength ratio as low as 0.23. As regards the upper limit, it is important to note that severe internal microcracking has been reported to take place in a concrete compression specimen at a stress-strength ratio of 0.4 to 0.6 / 275 /. It is thus not surprising that hence the cracking has accelerated, the creep behaviour also changes. It is also possible that the onset of cracking

depends on the degree of heterogeneity of the concrete; for instance, mortars are more heterogeneous than concrete containing a large-size aggregate and exhibit proportionality between creep and stress-strength ratio up to a higher limit of 0.85, when the strength is measured on slender creep-type specimens. The upper limit of the proportionality of the relation rises also with the duration of load. Jones and Richart / 286 / reported after their investigations the relation for one-minute under load and the graph is as presented in A-17 . Jensen and Richart in 1938 / 211 / also carried out short term tests on the proportionality of creep to stress-strength ratio up to a value of about 0.6 and above. They observed that there is an increased creep at higher ratios. On the average, the measured values is as shown below.

Stress-strength ratio	0.2	0.4	0.6	0.7	0.8	0.9
Creep (arbitrary units)	0.2	0.4	0.6	0.83	1.23	2.06

Above the limit of proportionality, creep increases with stress at an increasing rate. For stress-strength ratios below 0.7, Jensen and Richart observed that it is possible to approximate the curve by straight lines their intersection would

possibly representing the advent of cracking. The point of intersection would rise with time under load as the "cracking stress" rises with an increase in strength of concrete with time. Due to this reason, the limit of proportionality between creep and stress-strength ratio rises with time under load.

A sufficiently high sustained stress produces time failure known as static fatigue failure. The necessary stress to produce this type of failure lies in the region of stress-strength ratio of 0.7 to 0.9 of the short-term static strength of concrete. One factor which makes a comparison of results of different investigation somewhat difficult is the type and shape of specimens on which the compressive strength is determined. The creep specimens are often comparatively slender prisms with a height to laterel dimension ratio of 3 to 4. In some cases, the same type of specimen is used for the determination of strength, in others, cubes or standard cylinders with a height to diameter ratio of 2 are tested. Since the strength of concrete specimens is a function of slenderness ratio, it follows that, if strength is determined on the creep-type specimens, the limiting stress-strength ratio for the proportionality rule will be apparently much higher than when cubes

or standard cylinders are used. It is reasonable to assume that it is not the strength of the actual creep specimen that is relevant but the inherent strength of concrete under uniaxial compression. This of course is not readily determined but the cylinder gives probably the closest value. However, it is important that the same type of strength specimen be used for a comparison of creep results on different mixes and by different investigators. Neville in 1966 carried out test on the relation between creep and stress-strength ratio for (a) mortars made with different cements, stored at a relative humidity of 95 percent for 60 days using two stress levels; (b) to check the general validity of the relation, results of tests on mortars made with two cements only but subjected to a wide range of stresses as well as mixes with different water-cement ratios ranging from 0.40 to 0.52 were observed / 285 /. The strength of the mortar was determined on slender cylinder of size 50 mm by 230 mm so that the stress-strength ratio is higher than if the strength were determined on standard cylinders. The results of the entire test is as presented in A-18 in the appendix.

It is important to note that Neville calculated his regression line as shown in A-18

for specimens of standard mix proportions and subject them to the same stress only. Thus the effects of varying either the water-cement ratio or the applied stress were not included in determining the "best line". Despite the above conditions, all the points are scattered to the same extent. This indicate that the effect of changing strength by the use of a different cement or by varying the water-cement ratio is the same as far as creep is concerned. In the same way, for a given stress-strength ratio, creep is the same regardless of how stress and strength have been varied, provided their ratio is constant.

The relation between creep and stress-strength ratio seems to be approximately the same for different relative humidities of the ambient medium, as long as no appreciable shrinkage takes place. This implies that the specimens must reach hygral equilibrium with the ambient medium before the load is applied. The approximate validity of the stress-strength ratio rule was shown by tests carried out by Sheikin and Baskakov in 1956 /286, 352 /. Sheikin and Baskakov measured creep of concrete made from three different cements subject to stresses representing the same proportion of the ultimate strength. They observed that,

while the specific creep strains of the three concretes varied widely; 2.3, 5.2 and 3.3 x 10^{-6} per kg/cm^2 at 20 days, the total creep strains for the same stress-strength ratio were approximately the same. The total strains were 300, 330 and 300 x 10^{-6} after 20 days under load and 1240, 1530 and 1220 respectively after 660 days under load. The concrete strength as measured by standard specimen at 28-days were 257, 124 and 180 kg/cm^2 respectively. The divergence in creep for longer periods under load may well be due to the different rates of gain of strength with time for the different cements.

In one of the U.S. Army Engineers investigations /382 / values of creep and the modulus of elasticity at the application of load were measured for sealed concrete specimens. The results were reported in 1958 without much analysis. Neville / 286 / analysed the results of the above investigation using the ACI Building code 318-63. By assuming density of concrete, the values of strength for the different values of E measured from the test were estimated. The applied stress in all the cases as used by the Army Engineers is 32 kg/cm^2. Hence the stress-strength ratio were calculated for all the corresponding E and creep

measured. The resulting relationship between creep and the stress-strength ratio after one year as obtained by Neville is as shown in A-19.. It can be seen that the linear relation holds good up to a stress-strength in excess of 0.8. Various other investigators carried out research on the linear relationship between creep and stress-strength ratio and it was found to hold true except few of the investigators. Some of the investigators include Klieger / 224 / in 1957 and his results is as presented in A-20; L'Hermite and MacMillian in 1968 / 236 /. However, the results of investigations as carried out by de la Pena / 112 / in 1959 and Kruml's test in 1965 / 286 / on very thin mortar specimens and on mortar over a year old respectively; do not show any proportionality between creep and stress-strength ratio. The result is however an exceptional one.

The proportionality of creep and stress strength ratio for light weight aggregate concrete was however investigated by Kruml in 1965. Kruml observed that the lightweight aggregate exhibited proportionality up to about stress-strength ratio of 0.6. The U.S. Bureau of Reclamation also carried out test on the proportionality on a very lean concrete which

one may term lightweight aggregate concrete /383 /. She observed that the proportionality between the creep and the stress-strength ratio existed up to a stress-strength ratio of 0.3, which was the highest value tested.

The creep data presented above in this sections were obtained from mortars and concrete which had reached hydral equilibrium with the surrounding medium prior to the application of the load. If however, concurrent shrinkage takes place during the application of load, the shrinkage introduces internal stresses which are compensated for by the stresses induced by the applied load /233/. For this reason, below a certain threehold stress, creep is no greater than the shrinkage of an unloaded companion specimen. This was also confirmed by tests carried out by Giangreco /144/ in which he observed that for stresses below 25 kg/cm^2, creep of a neat cement paste is inditerminate, being the same order as shrinkage. His specimens were stored at a relative humidity of 50 per cent. Similar behaviour was observed again by L'Hermite /236/ when he tested concrete loaded after 180 days, one year and two years. This suggest that a part of shrinkage is intimately related to creep and furthermore, that concrete responds to total strain induced in it and not separately to strains from different sources. Linear relationship was also

observed by some of these researchers between creep and concurrent shrinkage; and stress-strength ratio, both in normal weight concrete and lightweight aggregate concrete. In the lightweight aggregate concrete, Jones et al /218/ observed that there is a linear relation up to a stress-strength ratio of 0.6, while Petersen and Watstein /302/ carried out tests in which they measured concurrent shrinkage and creep. In their analysis, there is a clear indication of the relation between stress-strength ratio and time deformation (which is the loss of prestress due to creep and shrinkage) being linearly proportional. The stress-strength ratio in this case of test being the ratio of the prestress to the compressive strength of the concrete.

L'Hermite /233/ from his test presented data on creep of concrete cured in water, loaded in air at a relative humidity of 75 per cent for 1200 days, and then unloaded and finally replaced in water. Under such conditions of change in the ambient relative humidity at the time of application of load, considerable shrinkage must accompany creep. However, L'Hermite did not state the magnitude of shrinkage but measured the total time-deformation. On plotting the total deformation against stress for a constant strength, Neville /286/ observed that, the resulting curve passes through the origin and departs from

linearity only at high stresses. Similar tests by Mamillan show a linear relation up to a stress-strength ratio of nearly 0.75. A further indication of linear relation between the total time-deformation and stress-strength ratio up to a value of about 0.6, was observed by Le Camus' /232/ in his test. His concrete was stored at a relative humidity of 75 per cent. Le camus observed also that even though Linear relation exist the lines do not pass through the origin. He therefore concluded that at certain value of stress-strength ratio, there is no time-deformation. This value decreases with an increase in time under load. The relation is as shown in A-21. A similar observation was made by Neville /275, 286/ after analysing several creep data and suggested that when shrinkage takes place, creep does not occur as a simple addition but catches up with the shrinkage only after a time. However, L'Hermite and Mamillan after observing similar relationship concluded that it may be as a result of cement paste which hardens in a state of tension and the strain resulting from this internal stress is annulled by creep due to the applied load. Thus, it implies that as long as a sufficient strain is not attained, there is no measurable creep. However, Neville's final conclusion seemed more logical. He assumed that creep and shrinkage are strains and are

similar in nature and the occurrence of one of them satisfies the physical requirements of the other.

The stress-strength ratio seems to overshadow other factors in creep with respect to this subsection, though three different curing procedures, two types of cement, two cement contents, and two nominal strength were used in the tests. However, one must admit that while calculation of creep from a standpoint of the stress-strength ratio may be convient for design purposes, an understnading of creep requires a more fundamental approach which is based purely on the physical properties of the gel.

(d) RELATIVE HUMIDITY OF STORAGE

The influence of the relative humidity of the air in which the concrete sample is stored has been known for some time. It has been earlier observed that drying concrete creeps at a higher rate and reaches higher ultimate creep than concrete which remains wet or dry. Troxel et al / 377 / have investigated into the effect of relative humidity on creep and observed that the influence of the humidity on creep is large. For a relative humidity of 50 per cent, creep may be 2 to 3 times greater than a relative humidity of 100 per cent. This is shown in A-22. However, the effect decreases with an increase in the size of the specimen. Since concrete exposed to lower relative humidities shrinks, it may be possible that the magnitude is affected by concurrent shrinkage. However, this must not be interpreted to mean that shrinkage in itself influences creep but only that the two phenomena are influenced by a common process during drying

Thus L'Hermite / 233 / in his investigation considers creep to be covarient with shrinkage. This, he meant that creep does not add to shrinkage but combines with it with a minimum of action. It is important to note that creep - shrinkage interaction does not arise from the restraint of shrinkage by aggregate, or from any other multi-phase features of concrete since neat cement paste exhibits the same behaviour as concrete.

In considering the influence of relative humidity on creep, it implies that the equilibrium relative humidity with the concrete sample or, more accurately within the cement paste is higher than the ambient humidity. According to Neville / 286 /, when the concrete is in hygral equilibrium with the ambient medium, creep is assumed for present purposes, to be unaffceted by the level of humidity, except for the secondry effect of the moisture content on continued hydration and hence on the further rate of creep.

With regards to the techniques .of testing Rusch / 340 / observed that it is possible to prevent absorption of water by a specimen but not elimination of water. For this reason, he studied the influence of load on moisture condition

in creep recovery. One of the specimens was in equilibrium with the ambient medium, while the other was stored so that it could not absorb any water from the atmosphere. The later specimen had much reduced creep recovery. He therefore interprets this to mean that even very small changes in moisture conditions can produce considerable strains.

(e) INFLUENCE OF TEMPERATURE

Temperature is the second major environmental factor in creep. This effect is usually considered to be less important than relative humidity because the range of operating temperatures in majority of structures is small. Yokota and Kotera /406/ have observed in their prestressed concrete tests that more creep takes place in prestressed concrete beams in hot weather, even if it is accompanied by a high humidity, than when the air is cool and dry. In mass concrete structures, where the concrete undergoes a temperature cycle, even if the ambient temperature is constant, the problem of influence of temperature on creep is of importance. The design stresses both in nuclear pressure vessels and in mass concrete structure such as dams are generally low, thus the effect of time deformation may not be over-emphasized to enable a reasonable accurate prediction of creep.

As is in the case of humidity, it is important to distinquish between the temperature during the period of curing preceding the application of load

and the temperature while the concrete is under load. In the former case, the properties of concrete as far as hydration is concerned, are more affected and this may modify the creep behaviour in addition to the more direct influence of temperature on the actual creep process. The effect of reduction in viscosity of the deformable phase and that of increase in vapour pressure on increase in temperature are greater than the increased hydration in both sealed and unsealed concrete /334/.

TEST RESULTS.

One of the earliest tests on the influence of temperature on creep is that carried out by de la Pena. He observed that creep of 1:1 mortar specimens stored under water to be proportional to temperature in the range of 32 to 122^{o}F (0 to 50^{o}C) /112/. Serafin and Guerreiro /286/ carried out creep test on mass-cured concrete, loaded at the age of 3-day. They observed that the rate of creep at 45^{oc} is higher than at room temperaute (29^{oc}) during the first 4 to 5 days after application of load. There-after, the rate of creep is the same in both cases, so that a constant difference in the magnitude of total creep persists. This difference of $0.55x10^{-6}$ per kg/cm^{2} was observed by Theuer's tests on a wet-

cured and subsequently jacketed specimens stored at 45°c and 27°c /370/. The decrease with time in the influence of temperature on creep takes place also in flexure/392/. Extensive data on the influence of temperature on creep were also present by Hansen /176/ who tested 6-months old, pre-conditioned during 24 hours to subsequent temperature of storage under load. After three weeks, mortar at (40 and 60°c) exhibited respectively 2 and 3 times the creep of similar specimen at 20°c. From his observation and conclusion, it is likely that the effect would have been relatively less pronounced after a longer period under load /176/. The relation between creep and temperature is approximately linear in the range of 27 to 80°c as reported by Hannant /173/. He carried out the test in a sealed specimens for a period of 733 days under load. In general however, creep at higher temperatures tends to show a higher scater in the measured values than at room temperature but the ratio of the vaues of high temperature to that of room temperature stabilizes after about a year under load /286, 173/. For unsealed concrete, the rate of increase in creep with temperature becomes progressively smaller, but the ultimate creep of sealed concrete is probably larger. Systematic tests carried out on the influence of temperatue on

creep /333. 269/ show that at any stress-strength ratio, creep generally increases with an increase in temperature up to about 71°c and there after decreases. However, some irregularities were noticed within the first 21-day of loading, thus it is more convenient to relate temperature to increase in creep beyond 21 days of loading. Some data obtained from the test is as shown in A-23. It /269/ was observed that the rate of creep at 71°c is about 3.5 times that at 21°c and above 71°c, the rate drops of steadily until at about 96°c, it is approximately the same as that at about 52°c which is about 2 times that of 21°c. Tests by Nasser and Neville /269/ confirming the results as stated above are presented in A-24 and A-25. It was also observed by Nasser and Neville that after 21-day under load, creep is linearly related to the logarithm of time under load, however, at 96°c there seems to be some irreguarity after 180-days. For the first 21-day, the logarithm relation is also completely different and less regular but the irregularity is much smaller when the concrete is heated at the age of 24 hours prior to loading. They also observed that concrete stored at 21°c, (that is one which has been at the same temperature throughout its life) shows a linear creep-log time relation as from about 3-days onwards (A-26).

The influence of temperature on creep is greater when the elevated temperature is applied a week before the application of load than when the higher temperature acts throughout the life of concrete /286/. The ratio of creep at 71°C to that at 21°C for concrete at a higher temperature throughout its life is 5.5 compared to 8.5 for concrete heated a week before loading /286, 269/. Similar behaviour observed with concrete loaded for 225-day under load. The data is shown in A-27. Here the ratio of creep at 71°C to creep at 21°C is about 2.4 in specimens permanently stored at the indicated temperature and as much as 4.0 when the temperature is raised only a week before loading. The most important conclusion to be drawn from the tests of Nasser and Neville /269, 286/ is that creep is not a monotonic function of temperature rather, it passes a maximum in the vicinity of 71°C. This was also confirmed by tests carried out by Marechal /252/. However, the influence of the sequence of heating and loading was also studied by Hansen and Eriksson /181/ who used flexural mortar specimens. They observed that heating before loading lead to lower creep than heating after application of load. The results as presented by Hansen and Eriksson is as shown in A-28. All the specimens they used were permanently submerged so that any influence of relative humidity was absent. It is possible that the observed difference is due to the

effect of the temperature change during the heating while under load per se, a case similar to the effect of a change in relative humidity /286/. An interesting practical experiment of increase in creep with temperature was performed by Ross et al /334/. They subjected a uniformly prestressed sealed beam to a temperature graduent ranging from 58°c to 86°c as shown in A-29 . It was observed that the deflection induced by initial heating was gradually reduced to one-third of its value because of the higher creep of the hotter part of the beam. Various other researchers carried out tests on the influence of temperature on creep. Day and Gamble/105/ researched into the effect of thermal pretreatment on the creep of hardened cement paste. In their test, miniature, saturated, hardened cement paste specimens were pretreated for 2-days at various temperature in the range of 20°c - 92°c. Specimens were then loaded for 6-days and unloaded. The immediate, creep and recovery strains were measured, as well as changes in microstructure using polysilicate and nitrogen sorption analyses. They observed that for both young (28 days) and old (2.5 years) specimens, thermal pretreatment resulted into significant changes in the microstructure

of the specimens; polysilicate content and average pore size increased while surface area decreased. In the young specimens, the pretreatment resulted in a reduction in subsequent creep deformation and good correlation was found between creep and the polysilicate content at the time of load. The results for the old specimens however is different because pretreatment resulted in an increased creep response and no correlation between creep and micro-structure changes.Polysilicate measurements made before and after loading showed significant differences. In the young specimens polysilicate content increased during loading while depolymeri-sation appeared to result from loading the old or matured specimens. Thus in Day and Gamble's test /105/, it appears that changes in the micro-structure of hardened cement paste can have profound effect on creep behaviour. This changes in micro-structure as a result of heating (increase in temperature)which influence creep can be categorised into two ways. The first is the micro-structural changes which occur before loading. Here, the creep of the hardened cement paste is a history-dependent phenomenon; it is a function of the state of the material at the time of loading, this state, in turn, is determined by the hydration, ageing and any thermal and hygrometric

pretreatments which change the structure of the material prior to loading. In the second category, micro-structural changes which occur during load leading to alteration of the micro-structure of the hardened cement paste under load results in an acceleration of creep behaviour; hydration during load produces an extra maturing creep components while carbonation, which results in a phase change under load causes a creep acceleration, a temperature rise under load produces large, irrecoverable, transitional thermal creep and it is well known that drying while under load produces a large drying creep component. The works of Illuston and Sanders /205/ tends to further confirm the above behaviour. Iluston and Sanders measured the creep of saturated mortar subjected to sustained torsional load. They observed that when the temperature was raised for the first time, the creep rate increased greatly in comparision with that of similar specimen heated before loading. No such change of rate was observed when the temperature was lowered or when it was raised to a level previously attained. Their results presented in graph is as shown in A-30. Two major creep components similar to that observed by Day and Gamble /105/ a difference of ten years interval were also observed. The first component is that of the constant temperature

creep which is said to be made of flow, an irrecoverable creep component with an important property of a given temperature; the rate being function of the maturity, but not of the age at loading and delyaed elastic strain, a recoverable creep component which occurs rapidly and reaches a limiting value which, for a given temperature, is reasonably constant regardless of the maturity; the rate of the development declines as the mortar becomes more mature. The second component is what they called Transitional Thermal Creep. This component occurs rapidly during and immediately following a rise in temperature to a previously unattained level. It occurs neither when the temperature decreases nor when the temperature is raised to a particular level for the second or subsequent time. The magnitude of the transitional thermal creep is approximately the same at all maturities. The characteristic of the components are as presented in A-30, A-31, A-32 and A-33. However, the age of cement mortars used by Illuston and Sanders /205/ were not more than 60-days old and the the temperature range is between 20 to 95^{o}C.
The works of various other researchers with respect to the effect of temperature on creep include Bazant et al /49/, who carried out Triaxial moisture control creep tests on hardened cement paste at high temper-

ature in the range of 100 to 300°c. They observed that axial creep of sealed specimen (with zero moisture loss) at temperature of 200°c is higher than axial creep of unsealed specimen exposed to hot water at the same pressure. The volumetric creep is about 50 per cent of the axial creep at room temperature. At a temperature range of 100 to 300°c, unsealed specimens that are loaded and heated at the same time, creep in air much more than those loaded two hours after heating. However, the difference in creep rates is negligible within ten hours of creep. Creep rate of unsealed specimens in air at 200°c is much larger than that of 100°c.

Others /28/ observe that the stress-strain curve for concrete at high temperature can be derived from stress-strain relation at room temperature and that the modulus of elasticity appears to decrease approximately linearly with temperature especially above 100°c.

On the other hand, some /49/ felt that the drying creep of heated concrete strongly depends on the movement of water, which itself is a rather scatter phenomenon. As such highly acurate predictions are impossible without a good model for the diffusion of water.

2.4. 4 MECHANISM OF DEFORMATION

In arriving at an understanding of the phenomena involved in creep of concrete or material, a discussion on its mechanism is of utmost importance. However, one must accept that such discussion is difficult as our knowledge of the actual problem is still inadequate. A number of theories have been proposed but none is capable of accounting for all the observed behaviour of material under creep. However, each explains a number of observations and accords fully with some or order of the experimental results. It may be possible that the actual creep involves two or more mechanisms, but before accepting such a combined theory, further tests and verification is necessary. On phenomenological basis, several broad mechanisms of creep can be distinquished. These include: Mechanical deformation theory, viscous flow, plastic flow, seepage of gel water, delayed elasticity and microcracking.

(a) MECHANICAL DEFORMATION THEORY

Freyssinet /199. 286/ attributes the behaviour of concrete under load to the internal stresses set up as a result of the change in the capillary structure of cement paste due to load. He assumed that under a compressive stress, the capillaries

are deformed and the water meniscus displaced outward to a point where the capillary diameter is large enough so that the tension under which the capillary water is held is decreased. This reduces the induced internal compressive stress and partially offsets the applied external stress. Thus, hygral equilibrium is upset with the result that water will evaporate from the capillaries until the internal vapour pressure reduced to the ambient value. During this process, the tension in the capillary water rises, and to maintain equilibrium, the compression in the solid phases increases. The resultant deformation under this action is said to constitutes creep. Thus one can said that it is in fact a delayed elastic phenomenon /286/, the delay being as a result of the time-lag in re-establishing vapour pressure equilibrium between the cement paste and the ambient medium. On removing the load, the resulting changes in the pressure differences between the water and air phases within the capillary structure create forces which tend to return the capillaries to their original shape.

The problem with this mechanism is that creep is completely reversible. The movement, both on application of load and on its removal, is elastic but is delayed by pressures that develop in the capillaries. This hypothesis cannot also explain

creep in water and it is not accepted nowadays. Later Freyssinet /140, 139/ expressed the opinion that the presence of load on concrete "increases the probability of rearrangements which lead to a reduction of volume of concrete". Thus one part of the creep is due to the tendency of the hydrated cement paste towards maximum stability under a sustained load, which is termed irrecoverable creep, while the other part consist of "elastic deformation deferred by the wetness of the concrete due to pressure" and is believed to be reversible. In other words, creep is supposed to be due in part to the surface tension effect and in part to the tendency to maximum stability. While part of the irriversible creep pastulated may be true, the theory does not explain the influence of the various other factors on creep and infact, there is no experimental evidence. However, Hope and Brown /198/ felt that in order to arrive at a definite explanation of this particular mechanism, of creep, measurements must be made of the physical changes which occur in the solid and in the pore structure as a result of applied stress and changes in humidity. If the creep mechanism involves the displacement of water, it is possible that the nature of the mechanism may be revealed by

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measurements of length and weight change under load. On the other hand, if the mechanism does not primary involve the actual displacement of water as suggested by other mechanism, then density and porosity changes should be measured. Thus Hope and Brown /198/ carried out an experimental program in which the relationship between length change, weight change and water content was investigated for cylindrical cement paste and mortar specimens subjected to uniaxial compression. They observe the following:

(a) Complete drying of specimens subjected to uniaxial compressive stress induces deformation in excess of that obtainable by shrinkage alone;

(b) Specimens from which all evaporable water has been removed exhibit no creep.;

(c) Creep resumes with re-absorption of water and, at the same stress level creep strain finally attains approximately the same limiting value on re-saturation irrespective of whether the specimens have been dried directly, subject to basic creep or stored at intermediate relative humidity before complete drying;

(d) Appreciable creep occurs with the macropore water frozen at -18°c, indicating that the water responsible for creep is in a more highly organised state than liquid water;

(e) No significant differences in water content due to creep can be detected under normal basic or drying creep conditions;

(f) Prior removal of macropore water reveals that the rate of loss of water is diminished by compressive stress due to closure of micropore spaces;

(g) The rate of re-penetration of water into the pore system after complete drying decreases with increasing stress level, and a greater irreversible weight loss occurs after complete resaturation;

(h) For previously dried specimens when water enters the smallest micropores, unit expansion against load requires 1.25 times the weight of water compared with unloaded specimens. From their observations, one cannot fully substantiate for this mechanism as they also observed that the water responsible for creep is in more highly organised state than liquid water. The organised state is however, not explained by Hope and Brown /198/. Knowing these facts themselves, they went

further to perform more tests now with the measurement of the physical changes in the solid and pore structure as a result of applied compressive stress and changes in humidity rather than the previous measurements. After their measurement and analysis, they observed the following once again.

(i) Hydration is inhibited by compressive stress;

(j) For specimens completely dried under load, the interlayer volume decrease compared with unloaded specimens. The density of the solid phase measured by methanol is increased by compressive stress, indicating that the solid volume including interlayer spaces consolidates under load. The density of the solid measured by water decreases indicating that the solid volume inaccessible to water increases with increasing stress level. Combining the above observations lead to the conclusion that some interlayer space is produced from larger micropores as a result of compressive stress but a greater volume of existing interlayer space is collapsed by the severe drying combined with compressive stress.

(k) Under basic and drying creep conditions the density of the solid phase measured by methanol is decreased by stress, indicating a decrease in porosity. An increase in the density of the solid measured by water reflects an increased weight of water in the pore system. The apparent interlayer porosities are also greater for the loaded specimen, indicating that interlayer space has been created from larger micropores. Increased weight of water is retained in these interlayer spaces compared with unloaded specimens. Based on their previous and present observations, a mechanism of creep which is a modified mechanical deformation theory was proposed based on the fact that urder sustained stress, interlayer volume increases as a result of the physical distortion and general reduction in spacing of the solid sheets of cement hydrate which originally formed somewhat larger micropores.

This mechanism is described below. The physical or diagramatic representation of the mechanism is shown in A-34. According to this mechanism, when compressive stress is applied to the specimen, hydrated cement paste exhibits a deformation which reflects the composite

modulus of the solid phase and the quasi-solid interlayer water. That the surface effects are involved in the interaction between stress and the energy with which the interlayer water and adsorbed water is bound to the solid, thus making the time-dependent deformation a gradual process. Each layer of crystal deflects under sustained load causing a reduction in the layer spacing. Existing interlayer spaces are reduced but additional interlayer space is created from the deflection of lamina originally spaced somewhat further apart. The net result is an increase in the volume of interlayer space in the specimens not completely dried under load. Thus there is a corresponding in ease in the weight of water retained in the micropores compared with unloaded specimens. Partial drying induces movment of this interlyaer water and accelerates the rate of creep. The reduced expansion of specimens (irreversible deformation) subjected to compressive stress on raising the ambient humidity should not be confused with the increased creep which occurs on drying. Severe drying under load results in collapse of a portion of the water braced structure. The quantity of collapse structure increases with increased compressive stress. The irreversible effects result from the inability of water to re-penetrate a significant portion of the collapsed structure. One wonders what happens to the water in the collapsed structure since in the previous

observation, it is a highly organised state than liquid water. The work of Day and Gamble (105) is similar to that of Hope and Brown /198/ but no hypothesis was given. However, Bazant /37/ after reading through this modified mechanical deformation theory felt that while irreversible changes in pore surface areas due to changes in water content and to desorption - sorption cycles have recently been directly evidenced by X-ray scattering measurements, the analogous changes of pore structure due to sustained compression have so far been inferred only indirectly from hypothesis on creep mechanism and from the changes of mechanical properties due to creep. However, he / 37 / strongly believe that the results presented by the authors / 198 / is a direct demonstration of the changes he mentioned above and in particular of the fact that the fraction of pore volume occupied by the smallest micro-pores (interlayer space) is increased by creep. The mechanism that under sustained stress the solid sheets of cement hydrate undergo severe physical distorsion (bending) and general reduction in spacing of the sheet and especially conclusion (d) of Hope and Brown / 198 / is not acceptable to him / 37 /, though some of the physical behaviours take place, it does not explain creep mechanism because of the following

reasons: (i) The structure of the pore space must, of course, change, but it is difficult to imagine that the change could consist mainly in large bending deflections of adjacent sheets as shown in A-34. Bazant / 37 / believes that one difficulty in the proposed mechanism is of geometric nature and becomes apparent when an array of a number of parallel sheets with interlayer spaces is considered as shown in A-35. He believes that because the overall deformation of this array due to creep is known to be very small (less than 0.001), the large lateral deflection in one sheet would have to be accompanied in the adjacent sheet by an opposite deflection of roughly equal magnitude, which is impossible. On the other hand, if a displacement which is small relative to the spacing of the sheets were assumed, contrary to the authors /198 / concept, the accessibility of pores to methanol could not be significantly altered but remain the same because the surface forces require a large displacement if they are to change significantly. Large displacement of sheets could be possible on the end segments sticking out of the array of parallel sheets, however such loose segments cannot receive any significant portion of the microscopic compressive load. Bazant / 37 / believes that an open

configuration such as that shown in A-34 by the authors may exist in the parallel array as shown by "A" in A-35./ If closing of such open configurations should result in small microscopic strain, then these configurations would have to be spaced mutually very far apart, and their closing under compression could not add significantly to the micropore volume. (2) By means of large lateral deflections of sheets, it is also difficult to explain various other phenomena. For instance, when a fuly dried concrete is considered in which no closing of pore spaces occurs because there is no creep, why then, after subsequent rewetting the pore spaces would close under load, as is known by the fact that creep which exceed the swelling on the previous rewetting takes place. Or when the full water saturation is attained, that implies the closing of the sheets because water has no empty space to go to and must compress in bulk. Then, why the creep at full saturation about the same as creep of sealed specimens, (according to the authors) which are not quite saturated. Furthermore, if closing of interlayer space under load is the source of the additional compression creep due to drying (as compared with basic creep), why is it then that drying causes an additional creep also in shear or tension in which roughly half or nearly

all of the interlayer spaces should have to get opened rather than closed. He also wondered why not only a decrease but also an increase of water content causes additional creep in compression, shear or tension. If this mechanism of Hope and Brown / 198 / is correct, why is it that any change of water content causes an additional recovery of compression creep as demonstrated by experiments / 37, 198 / carried out by himself, and any change of temperature causes an additional creep. However, Bazant / 37 / strongly believe that the important conclusion regarding pore structure changes under sustained stress by the authors / 198 / can be alternatively explained by the hypothesis that creep is caused by migration (diffusion) of certain components of solids (probably Ca - ions) between load - bearing and load - free domains in the micro-structure. This mechanism was proposed by Bazant himself in 1973 / 37, 198 /.

Thus in Bazant's opinion / 37 /, he does not believe in Hope and Browns proposed creep mechanism rather, the hypothesis of diffusion of components of solids coupled with diffusion of water along micropores seems to be a preferrable explanation for the phenomena reported by the authors and can be regarded as an additional confirmation of his own hypothesis.

In replying to the above observations by Bazant, Hopes and Brown / 199 /, though accept most of the explanations by Bazant, believe that they cannot envisage that the coupled diffusion theory can adequately account for the totally consistent and repeatable measurement of an increased weight of water retained in the interlayer spaces of creep specimen :. Thus according to them, their results invalidate all previous creep theories and is an important and strong argument favouring their proposed model.

Latest work on this mechanism is by Rossi and Acker / 335 /. They proposed that the application of the instantaneous force in the basic creep or relaxation test in compression leads to the formation of a number of microcracks distributed throughout the volume of the specimen. Statistically, the largest dimension of the microcracks propagated in the mortar along their length is at a very small angle to the direction of loading which is vertical. The microcracking is revealed by the abundant acoustic emission recorded during the instantaneous loading. This is shown in A-36 . On proposing their mechanism, they think that the formation of these microcracks constitutes, for the medium - concrete, a suden disturbance of thermodynamic equilibrium, since at the initial

time, all points in the medium are almost in a state of thermodynamic equilibrium. The hydric shock resulting from the formation of the micro-cracks is, locally much more intensive than the shock produced by the removal of the specimen from the mould (drying of concrete). The gradients of concentration of water molecules that result frem the microcracking induce movements of water vapour from the micropores to the microcracks, leading to drying of the micropores. This drying of micropores is termed "Post-cracking" drying which cause shrinkage that is of the same type as the normal drying shrinkage. If the existence of "post-cracking drying shrinkage" is accepted, then the microscopic mechnaical behaviour of concrete in the context of basic creep or relaxation can be easily explained. Thus, during basic creep test, deformation induced by post-cracking drying shrinkage is added to that resulting from instantaneous loading, since the mode is a constant imposed compressive stress. On the other hand, in a basic relaxation test, since the imposed microscopic deformation is constant, the deformation induced by the post-cracking drying shrinkage, which has the same sign as the deformation resulting from the instantaneous compressive loading, will necessarily lead to a gradual reduction of the compressive load. The schematic diagram is presented in A-37.

The compressive stress applied in the basic creep and relaxation tests, substantially diminish the large tensile stresses induced in the vicinity of the free drying surfaces. In the case of basic creep or relaxation, these free surfaces are the kinematic discontinuities constituted by the microcracks. Thus it must therefore lead to delayed microscopic deformation which are observed experimentally. Secondly, in the physical process proposed, the larger the instantaneous stress, the greater the density of the microcrack formed and therefore the higher the basic creep or relaxation rate. This also agree with experimental results in literature.

(b) PLASTIC THEORIES

In this mechanism, it has been suggested that creep of concrete may be in the nature of crystalline flow. That means that it is as a result of slipping along planes within the crystal lattice. Thus plastic deformation is the result of slip along the plane of maximum shear in the crystal lattice. Under such circumstances, no volume change takes place.

Lynam / 243 / felt that concrete is several times stronger in shear than in tension, as such

it would be expected to fail in tension before a shear slip occurs, thus the occurance of creep in tension cannot be substantiated. The second problem with this mechanism is that creep of concrete leads to a definite reduction in volume. Glanville and Thomas / 160 / further modified this mechanism by saying that creep at low stresses may be viscous and at high stresses in the form of crystalline slip. The problem with this modified form is that one can argue that creep-time relation for low and high stresses is of the same general form and no fundamental change in behaviour is apparent. Though, at high stresses the deformation of concrete resembles somewhat plasticity, and it may be reasonable to use the theory of plasticity as a basis of ultimate strength design of structures, one should bear in mind that there is a fundamental difference from the behaviour of metals. While metal's broken bonds continually re-establish themselves by virtue of strong cohesion, concrete forms micro-cracks more readily and once the bond between the aggregate and paste have been broken, no mending process may take place again. Even though some form of the plasticity may enter the deformational behaviour of concrete at stresses nearing failure, this is not of great significance to the creep behaviour of concrete under normal loads.

Further work on this mechanism to really understand the phenomenon and modification of it were carried by Mindess et al / 259 /, Bentur et al / 58 / and Bentur et al / 59 /. After a set of three experimental works which involve measuring the relationship between creep and shrinkage of pure calcium silicate pastes and the materials, specimen preparation method and mechanical test results; Secondly by the determination of pore structure and surface area of pure calcium silicate pastes by adsorption of water and Nitrogen, and Helium inflow tests; and finally by measuring irreversible strains of pure calcium silicates pastes which were loaded without drying, dried to 53 per cent relative humidity without loading and were loaded and simultaneously dried to 53 per cent relative humidity; and correlating structural changes occuring within the hydrated pastes with the microstructural changes in the paste, they arrived at a modified plastic theory which is almost a middle-way between micro-crack theory and plastic theory. The modified plastic theory states as follows (1) It is assured that the components of the hydrated paste that are responsible for irreversible strains are the pore component (pore sizes greater than 26 Å in diameter) (2) the response of well-hydrated pastes is different from that of pastes with

relatively low degree of hydration. Changes in the calcium silicate hydrate component are responsible for most of the irreversibility in well hydrated pastes, while changes in the pore component are most important for young pastes. (3) Indications from nitrogen adsorption data suggest that when young paste are dried a reduction in the content of pores in the range 40 to 100 Å diameter occurs, and that this reduction causes a decrease in Nitrogen surface area and may result in irreversible strains. Adsorption of water vapour does not show this effect. (4) In mature pastes loading produces structural changes that are fundamentally different from the structural changes that occur on drying, suggesting that the basic creep mechanism is fundamentally different from that of drying shrinkage. (5) An increase in the surface area calculated from Nitrogen adsorption is observed on loading mature pastes and is explained by a concept of micro-shearing between calcium silicate hydrate particles. (6) When mature pastes are dried irreversible strains are thought to be caused by the formation of additional silicate bonds within the calcium-silicate- hydrate component. The changes in bonding are modified by the super position of an external load.

Thus, this mechanism accept that basic creep is caused by micro-shearing (crystalline slip) between calcium silicate-hydrate particles.

(c) VISCOUS AND VISCO-ELASTIC FLOW THEORIES

Viscous flow as a mechanism of creep was first postulated by Thomas / 372 / who considers concrete to consist of two parts: cementitious material which behaves in viscous manner when loaded and an inert aggregate, which does not flow underload. When concrete specimen is loaded, the cement flow is resisted by the presence of the aggregate, and as a result of this resistance the aggregate becomes more highly stressed while the stress on the cement paste decreases with time. Since, the creep of cement paste is proportional to the applied stress from this mechanism, the rate of creep will be progressively reduced as the load is transferred from the viscous to the inert material, hence Arnstein and Reiner / 21 / infer that the rate of creep should depend on the nature of the cement paste but not on the properties of the aggregate. This is in contrast with the constitutive model. True viscous flow require a constant volume, while axial creep of concrete results in a lateral creep corresponding to a creep Poisson's ratio well below 0.5. If the

flow were viscous, it would be necessary for the gel to flow into pores within the hydrated paste. There is no evidence of flow into the capillary pores. Furthermore viscous flow requires a proportionality between stress and strain and between the rates of stress and strain at any stress. This condition is not completely true in the stress-strain relation in creep or elasticity test.

Hansen / 176 / however, felt that the modulus of viscosity as well as the delayed elastic modulus of concrete increase with the amount of cement gel formed. The two moduli must also increase with a decrease in the distance between cement grains, that is to say with a decreasing water-cement ratio. Thus Hansen / 176 / believe that it is the same factors that influence strength, elasticity and deformation under sustained stress. How these factors influence the deformation is however not detailed. Further work on this mechanism were carried out by Roy and Asaga; Shaughnessy and Clark / 192, 351, 336 / who investigated the effect of mixing on the flow property of concrete. They observed that in the absence of admixture in fresh concrete, shear, stress-shear rate relationship reveal a change from plastic flow to viscous flow with increasing

shear rate. That yield stress and plastic viscosity decrease substantially with increasing intense mixing.

(d) SEEPAGE THEORY

Before going into explaining the seepage theory it is worth mentioning the Elastic after-effect theories by Maney / 249 / which deny outright the existencé of creep. He proposed that there is no creep at working loads, rather the effect on loading being simply an elastic change due to the change in non-uniform shrinkage. This theory has been disproved by experimental results obtained as a result of creep. The second theory w th mentioning is the solid solution theory by Lea and Lee / 231 /. The theory though accepts the existence of creep, one feel is similar to the mechanical deformation theory.

The seepage theory of creep of concrete was first postulated by Iynam / 243 /. Seepage is regarded as a viscous flow of the dispersion medium through and ahead of the disperse phase. In a simpler term, this means in a hydrated cement paste which is a rigid gel, load causes an explulsion of the viscous component from the voids in a redistribution of stresses from the viscous component to the elastic skeleton. Thus creep in

concrete is taken to be due to seepage of gel water under pressure. Removal of stress from such a mixed system is characterized by a partial immediate recovery, followed by a slow elastic after-effect as seepage is reversed. There is thus a volume decrease under a compressive stress in contrast to plastic flow and viscous flow which involve no change in volume. In this way, creep is analogous to shrinkage but the motive force is an externally applied pressure instead of a direct vapour pressure differential with the ambient medium as is in mechanical deformation theory and the removal of capillary water has no effect on deformation under this mechanism. Thus in seepage theory, creep is a manifestation of the delay in re-establishing the equilibrium between the gel and its surroundings as water is squeezed out as a result of stress. The creep recovery is simply the tendency to re-establish the original state of affairs when the external load has been removed. However, full recovery is prevented by the formation of new bonds when the gel particles are allowed to come closer to one another as a result of creep. Thus a new stable position of the gel particles is established. However, it should be stressed here that it is only the gel water that is involved in the seepage

movement and not the capillary water nor the chemically combined water enter the picture.

It would therefore appear that shrinkage due to the loss of moisture and creep due to seepage are interelated phenomena since this moisture movement can be produced by external pressure. Nevertheless, the seepage theory of creep has had a considerable support because of the similarity between cement gel and rigid gels and partly due to the fact that the theory gives an explanation of the large creep of drying concrete with wet or dry state. However, the seepage theory does not explain why concrete dried in air and then subjected to compressive stress in water creeps twice as much as concrete stored and loaded in water. Furthermore, the creep of concrete under tension in water is about the same as under compression in water, while to accord with the seepage theory it should be larger under tension. Other set-back arises from the fact that the measured loss of water from concrete under a compressive stress is insufficient to account for the quantity supposed to be lost by seepage. It is also worth mentioning here that substantial creep in mass concrete where no loss to the outside is possible has been measured / 286 /. While the seepage

theory may not be completely correct, there is no doubt that the presence of evaporable water in the hydrated cement paste is a necessary prerequisite for creep to take place. Indirect evidence of the role of water in creep is given by (A-38) which shows that mortar stored in mineral oil is not subject to creep while water stored mortar is subject to creep / 286 /. It was also found / 174 / that liquids with large molecular size have no effect on creep but methyl alcohol has some effect. Since a molecule of methyl alcohol is 3.5 $\mathring{A}$ in diameter, approximately the same as water, this would suggest that the molecular size is the significant factor.

(e) CONTRIBUTION OF MICROCRACKING TO CREEP

It is important to note that microcracking is responsible for only a part of the deformation associated with creep. This part caused by microcracking is the irrecoverable part. The role of the interface between aggregate and the hydrated cement paste is that of providing a discontinuity when micro-cracking takes place and the type of crack considered here is known as bond crack. The extent of development of the bond cracks due to creep depends on the quantity of bond cracks in existence prior to application

of load. Thus, generally, the later a given load is applied in the life of concrete the lower the creep / 286 /. It is also known that sealed concrete creeps initially more because of the absence of bond cracks caused by shrinkage prior to the application of the load. Though, after a time, the rate of creep of sealed concrete may fall below that of similar unsealed concrete, the sealed concrete has less water present for contributing to creep. However, the role of micro-cracking is considerably greater at loads nearing ultimate. It has been observed / 255 / that while shrinkage, short-term load and creep all cause an increase in microcracking, for the same strain the increase in micro-cracking is in a decreasing order for shrinkage, short-term load and creep. The development of microcracks associated with creep has been observed and confirmed by noise measurements / 323, 340 /. Neither elastic nor viscous deformation produce noise because there is no microcracking, since movement occurs at molecular level only. However, when crack is formed or extended in case of existing one, the release of energy produces an elastic wave so that noise is associated with disruption of internal structures. The noise as measured using a device on the surface of the specimen is shown in A-39 / 340 /

(f) POSSIBLE MECHANISM AS A RESULT OF ADMIXTURES

After some investigations using hydroxylated carboxylic acid and liqnosulphonic acid as admixtures / 212 /, it was concluded that the mechanism through which admixture influence creep is still uncertain and the possible mechanism should be further studied. However, further work on these area / 60, 91, 98, / indicate that increased creep and shrinkage strains observed in pastes containing $C_a Cl_2$ is believed to be due to an enhanced contribution of changes in pore size distribution within the pore component. The increase in creep and shrinkage strains were observed for both reversible and irreversible components. However, the work of the former investigators / 212 / were mainly with surface tension of the mixing water, density or morphology of the gel and rate of loss of water as altered by the admixtures and not specifically the pore sizes as indicated by the later / 60, 91, 98 /.

2.4.5 EXPRESSIONS FOR LONG-TERM DEFORMATION

From the time it was realised that the progress of creep with time under load follows a definite pattern, attempts have been made to express this relation in the form of an equation. This is to enable the values of creep to be predicted without performing long-term tests. It is however not known

whether there is a finite limit to creep after infinite time. It seems that, while creep may continue for a very long time, it probably tends to a limiting value.

As far as creep expressions are concerned, two broad types exists: those which tend to a limiting value known as the exponential and hyperbolic expressions and those that do not have a finite limit known as the power and lagorithmic expressions.

All these expressions contains a varying number of constants or coefficients in which some are clearly emperical and others quasi-rational. Work by various investigators indicate that the hyperbolic expressions have been found to fit experimental values generally very well and are commonly used /286/. However, recent work by Brazant /42, 34, 39, 43, 44, 51, 53, 45, 47, 52 and 245/ indicate that power expression also fit experimental values very well after modifying and improving the expression. Having the mechanism of creep discussed in the previous section, one would review only the more important expressions of creep in this section.

(a) POWER EXPRESSIONS

TABLE 2 CREEP EXPRESSIONS

Equation Number	Expression	Reference
13	$C = Q\sigma^{A} t^{B}$	/362/
14	$C_{sp} = A\, t^{\frac{1}{B}}$	/350/
15	$C = A\left[\frac{t}{1400}\right]^{1/4}$	/141/
16	$C = A\sigma t^{1/3}$	/343/
17	$\varepsilon = \Psi(t)\,\frac{\sigma}{E_o}$	/343/
18	$\varepsilon_c = \left(\varepsilon_e\right)^{1.2} t^{c} g(\tau)$	/298
19	$J\left(t,t'\right) = \frac{1}{E} + C_o\left(t_1 t'\right) + C_d\left(t,t',t_o\right) - C_p\left(t,t',t_o\right)$	/34, 35, 39, 42, 43, 44, 45, 47, 51, 52, 53, 168, 219/

Equation 13 /362/

This equation was suggested by Straub /362/, and the term has the following meaning:

C = Creep

σ = applied stress

t = time under load

Q = a parameter depending on the properties of concrete.

A and B are constants depending also on the properties of concrete at the time of application of load.

Equation 14 /350/

This equation was proposed by Shank /350/. In this expression it implies that creep is proportional to stress applied.

C_{sp} = specific creep per PS1

t = time under load in days

A and B are constants. They depend on the quality of concrete, curing conditions and age at loading. Most of the time, the constant are determined experimentally. Some other times they are assumed.

Equation 15 /141/

This equation as suggested by Friedrich /141/ was an improvement on equation 14 proposed by Shank /350/. Here Friedrich assumed that the value of creep after 1400 days under load represents the limiting creep. The terms in the equation remain the same as that of equation 14 .

Equation 16 /343/

This expression was suggested by Saliger /343/. Usually the $t^{1/3}$ is expressed in the form ϕ (t). The expression is almost the same with all the above expression but the constants differ. The terms remains the same.

Equation 17 /343/

The expression was also suggested by Saliger or rather it was an improvement on an existing expression. The equation is based on the fact that it is customary in some countries to express the total deformation of concrete in terms of the elastic deformation on loading.

E_o = modulus of elasticity of concrete at application of load.

$\Psi(t)$ = creep ratio-ratio of total deformation after time t to the elastic deformation on loading.

The rest terms remain the same as above. To overcome the absence of a limit in the expression, Saliger /343/ assume that there is no increase in creep beyond the period of 30 months under load. This of course is not true. The expression as tested /286/ indicate that it cannot be used for loading ages greater than a few months.

However, one should point out that the secret of Bazant equations is from the Saliger's equation. He modified the equation by addition of factors and also the influence of various concrete composition, drying, environmental and shrinkage. Infact, he combined various equations into one equation known as double Power Law.

Equation 18 /298/

This equation was suggested by Parrott /298/. He formulated the equation on the bases that creep is independent of stress, porosity, moisture state and hydration but dependent on age at loading.

ε_c = creep strain

t = period under load

$g(\tau)$ = creep reduction function dependent upon the age at load.

Equation 19 /Bazant/

As earlier said, the origin of most Bazant equation is from Saliger's proposed equations. With the widespread of electronic computation, giving rise to more sophisticated formulations in design and detail analysis of available test data; Bazant developed a constitutive equation which is quite general and agrees well with wide range of experimental observed proporties of creep of concrete.

Since the constitutive expression in itself is in the form of differential or integral equations, he went further to put it in a simplier formulations consisting of algebraic formulas. Thus the meaning of the terms in the equation and the condition attached are as follows. Before the terms, it worth pointing out that his equations consist of five parts, namely: Shrinkage /39/, basic creep /44/ that means creep at constant humidity and temperature, creep at variable humidity /45/ and recently he modified the entire formula /52/. In the Bazant equation

the time-dependent deformations were characterised basically by the following concepts.

(1) Square-root hyperbolic law for shrinkage
(2) Double Power, law for creep
(3) Diffusion-type size dependence of humidity effects
(4) Additive drying creep term related to shrinkage and
(5) Activation energy concept for thermal effects.

Thus Bazant formulation is guided in many respects by previous work on the effect of composition upon creep and shrinkage and the works of Kesler, Neville, L'Hermite and Branson were examples of such previous work /43/.

Ther Term $C_o(t,t')$ /44/

$J(t,t')$ = Creep function = strain at time t caused by a unit sustained uniaxial stress acting since time t'

E_o = asymptotic modulus which does not represent an actual modulus for any load duration measured but represent merely the left-side asympotote of the creep curve in log-time. This asymtote is reached only at ultra-short times $=1.5 \times E_{static}$.

$C_o(t,t')$ = unit basic creep known also as the the specific creep. That means a creep without moisture exchange.

The terms $C_d(t,t', t_o)$ and $C_p(t,t',t_o)$

C_d = increase in creep strain during drying

C_p = decrease of creep strain after drying. That means in predried specimens.

The subscript d refers to drying and P to predried.

In order to deduce reasonable expressions for C_d and Cp, Bazant et al /39/ considered twelve basic facts on creep at variable humidity. The basic facts include:

(1) That in saturated conditions the creep equation is automatically changed to basic creep equation.

(2) Up to a certain time, the creep of drying concrete is higher than the basic creep.

(3) At the beginning of drying, the creep curve has the shape of power function or sum of power functions.

(4) For specimens loaded after the termination of drying, that means reduced and constant moisture content, the creep curves have the shape of power functions.

(5) Creep after drying is independent of the thickness of specimen

(6) The longer is the period of $t' - t_0$ from the start of drying to the instant of loading, the smaller is the increase of creep due to drying

t' = age at loading

t_o = age at which drying begins both in days

When $t' - t_o > \tau_{sh}$ - known as shrinkage square half-time, the increase of creep due to drying vanishes

(7) The thicker is the specimen (greater γ_{sh}), the smaller must be the increase of creep due to drying.

(8) For a sufficiently thick specimen ($\gamma_{sh\to\infty}$) the general creep equation (equation 19) changes to basic creep equation. That means only the first two terms is retained, and not only in condition one above.

(9) At higher age, t', the increase of creep due to drying must be less, similarly to basic creep.

(10 Concrete which exhibits higher shrinkage exhibits also a greater increase of creep during drying.

(11) The decrease of creep due to the lower moisture content after drying is not prop-proportional to shrinkage constant $\varepsilon_{sh\infty}$ when various concretes are considered and is approximately independent of $\varepsilon_{sh\infty}$(shrinkage constant)

(12) Unlike shrinkage, creep of saturated specimens and creep of sealed specimens (in which the the pore humidity drops to about 0.98) are about the same.

With the above twelve basic facts and the work of various previous investigators, Bazant et al , /39, 43, 45 / derived expressions to obtain values for creep factors C_d and Cp. The various main

components of creep as indicated by Bazant et al / 39 / include basic, drying, and predried creeps.

Bazant et al went further to improve the formulas of C_d and C_p by adding some parameters or changing the quotient of the expressions given the values of C_d and C_p / 45 /. Further improvement on the expression for basic creep C_o (t, t') was also introduced as a result of temperature effect on basic creep / 45 / , and as a result of temperature effect on drying creep; C_d (t, t', t_o) and C_p (t, t', t_o) / 47 /. The latest review of the BP model was the one that took account of the cyclic component of environmental relative humidity / 52 /. The shrinkage and drying cr p strains in the interior of a specimen cannot respond instantaneously to a change of environmental humidity. The true, unrestrained shrinkage and creep strain at a point can depend only on the pore relative humidity at that same point, and the effect of environmental humidity can only come due to the diffusion of moisture between that point and the specimen's surface. According to the diffusion theory, shrinkage should vary as the thickness square and be proportional to the drying diffusivity of moisture. Thus based on diffusion theory, a further factor known as the correction factor for

the cyclic humidity component is introduced in the equation for determining C_d, the additional compliance due to drying (increase in creep due to drying). One should stress here that the double power law by Bazant and Panula is being continually reviewed. This is done by Bazant and various other investigator by carrying out probabilistic and statistical analysis using experimental data with the aid of electronic computer. Often, changes are made or modifications are made by adding a factor or changing parameters in the basic creep / 51, 53/, using the equation proposed and analysing the uncertainty of creep and shrinkage using matrix method / 245 / and so on. One therefore expect further modification even tomorrow.

(b) EXPONENTIAL EXPRESSION

table 3 EXPONENTIAL CREEP EXPRESSION

Equation Number	Expression	Reference
20	$C = C_{sp\infty}\sigma\left[1 - \exp\left\{-A\left((t+B)^F - B^F\right)\right\}\right]$	/ 371 /
21	$C = C_\infty\ (1 - e^{-At})$	/ 253 /
22	$C_{sp} = F(k)\ (1 - e^{-At})$	/ 22 /
23	$C = S_\infty\ (1 - e^{-mt})\ V_{cp}\ \frac{\sigma}{\sigma S}$	/244/

24	$C_{sp} = C_{sp\infty}\left[1 - \exp\left\{ - A \log \frac{k + t}{k} + Bt \right\}\right]$	/ 234 /
25	$C_{sp} = A \exp\left\{ B\left[1 - \left\{\frac{k}{t}\right\}^{D}\right] \right\}$	/ 132 /

The terms of the Exponential expressions are explained below.

Equation 20 / 371 /

The equation was developed by Thomas / 371 /. He assumes that the creep of neat cement paste is proportional to the applied stress. According to him, the creep of hydrated cemert paste is restrained by the aggregate and the load is gradually transferred from the cement paste to the aggregate. Thus, the rate of creep is progressively reduced with time.

C_{sp} = specific creep at infinite time (the limiting value of creep)

A and F = Constants for a particular concrete

B = constant introduced because the rate of creep in a viscous material is not necessarily infinite on application of load.

In order to take care of the lateral resistance of the aggregate to the flow of cement paste, Thomas / 371 / assumed that the lateral movement of concrete is negligible. That transverse plane section remain plane, and that the aggregate is inert and does not creep. The practical value of this relation is limited as creep after one year under load has to be determined experimentally

Equation 21 / 253 /

This equation was proposed by McHenry / 253 / and assumed that, for a given concrete, the application of load creates a certain creep potential and the rate of creep at any time is proportional to the amount of potential creep still to appear, a property of visco-elastic flow.

C = Creep after time t.

C_{∞} = limiting creep

A = is a constant.

The McHenry's equation has not shown any good aggreement with experimental data / 286 /. For this reason, he went further to improve the relation thereby introducing five constants into it. Despite this effort, it was observed / 286 / that it is unsatisfactory when compared with experimental data for ages at loading smaller than 5 days and

in some cases inaccurate for periods under load greater than one year.

Equation 22 / 22 /

This expression was proposed by Arutyunyan / 22 /. He assumed that shrinkage is proportional to the cement paste content. The terms remain the same as explains in equation 21

$$F(K) = C_{sp\infty} \frac{D}{k} \; ; \quad D = \text{constant}$$

and K = age at loading

Equation 23 / 244 /

This is proposed by Lyse / 244 / who assumed that creep is proportional to the applied stress

S_∞ = Ultimate shrinkage (limiting shrinkage).

m = rate of shrinkage depending on relative humidity

V_{cp} = fractional volume of cement paste

σ = applied stress

σ_s = magnitude of stress which gives creep of the same magnitude as shrinkage at the relative humidity of storage.

This relation does not apply to creep specimen stored in water when shrinkage has a small negative value and yet positive creep is observed.

Equation 24 / 234 /

Proposed by L'Hermite / 234 / , the founder of RILEM who assumed that the viscosity of concrete has a component which increases with the age of concrete. The terms remain the same as the previous relations. However, it was observed / 286 /, that the term Bt is usually small and can be neglected.

Equation 25 / 132 /

This expression was suggested by Erzen / 132 / and the terms are as previously explained.

(C) HYPERBOLIC EXPRESSION

Hyperbolic expressions have been found to fit experimental values generally very well / 286 / and are commonly used. About four expressions are proposed by various investigators under this sub-section. It is summerised in the table below.

TABLE 4 HYPERBOLIC CREEP EXPRESSION

Equation Number	Expression	Reference
26	$C = \frac{t}{A + Bt}$	/ 326 /
27	$C = \frac{t}{(100 + t)B}$	/ 326 /
28	$C = \sigma \frac{Dt}{E + t}$	/ 240 /
29	$C = A + \frac{B}{t}$	/ 10 /

Equation 26 / 326 /

This was suggested by Ross / 326 / and

C = creep strain

t = time in days

A and B = constant determined experimentally.

Equation 27 / 326 / was a modification of that of 26 because Ross observed that A and B vary in step and to allow for discrepancies at early ages, he took A/B to be 100.

Equation 28 / 240 /

This expression was proposed by Lorman / 240 / independent of Ross, though the two equation seems to be similar. All the terms remains the same except that E is a constant and not modulus. Ali and Kesler / 10 / proposed that of equation 29 and the terms remains the same.

(d) LOGARITHMIC EXPRESSIONS

The only logarithmic expression one could trace was that of U. S. Bureau of Re-clamation / 380 /. In the equation, it was assumed that the rate of specific creep is inversely proportional to time. Considering the elastic and creep strains together, they arrived at an equation below.

$$\varepsilon = \frac{e}{\sigma} + F(k) \log_e (t + 1) \qquad (30)$$

where ε = total strain (elastic + creep)

e = elastic strain

σ = applied stress

The rest term remain as before.
The equation is used for stress-strength ratio not exceeding 0.35. Good agreement with experimental data has been obtained, at least for mass concrete

used on the Bureau of Reclamation projects. However, creep values for short periods under load depart from the straight line.

The value F(k) is a property of a particular concrete.

(e) Indirect Power Expression

The creep expression classified as Indirect power expressions are those expressions which are multiples or additions of some parameters which are obtained using power expression. However, rather than expressing the parameters in power form, the values are obtained from graphs. Thus one can use the expressions without really knowing that they are power expressions. Most of the expressions are given by CEB - FIP and ACI. It is an attempt to simplify the creep expression. They are summerised in the table below.

TABLE 5 INDRECT POWER EXPRESSION

Equation Number	Expression	Reference
31	$\varphi = \varphi_o \; \varphi_1(\tau) \; \varphi_2\{t - \tau\}$	/ 368 80, 40/
32	$J(t, t') = \frac{1}{E(t')\left[1 + C_1 C_2 \lvert t' \rvert^{-0.118}\right]} \cdot V_{t''} \; C_u$	/ 73, 239 18, 40 46 /
33	$\Sigma_k = \frac{\sigma_c}{E_b} \phi(t)$	/ 80,116 /

Equation 31 / 80, 368 /

This is proposed by CEB-FIP and the terms means the following:

φ = creep coefficient

φ_o = coefficient which depends on the environmental conditions, the concrete composition and the equivalent thickness of the member

= $\beta_1 \; \beta_3 \; \beta_4$ obtained from graph.

$\varphi_1(\tau)$ = β_2 - depends on the degree of hardening - also obtained from graph.

$\varphi_2 (t - \tau) = \beta_5$ - depends on the time since the application of load.

The graphs are presented as appendix A-40, (a), (b) (c) and (d).

Equation 32

This equation is proposed by ACI committee 209 based on the work of Branson et al / 18, 239 37 /

$J (t, t')$ = creep compliance which is a function of time t and age at loading t'.

C_1, C_2 = Constants which depends on ambient humidity and member dimension.

$V_{t''}$ = Creep coefficient after t" days of loading.

C_u = Ultimate creep coefficient which is defined as the ratio of creep strain at infinite time to initial strain at loading. This coefficient is also a function of ambient humidity, minimum thickness of a structural member, slump, cement content, per cent of fines and air content.

Equation 33

This equation was proposed by Deutsches Institut fuer Normen. This is a German Code for prestressed concrete. This equation is similar to equation 31 and also found in CEB - FIP recommendation.

ε_k = Creep strain (deformation)

σ_o = constant stress

E_b = Elastic modulus N/mm^2 obtained from a table

ϕ_t = Creep coefficient obtained either from a table or from an expression given in the code with all parameters obtainable from a table.

(f) CREEP UNDER VARIABLE STRESS

Most expressions given in the previous sections on creep were based on conditions of a constant stress. However, in many practical cases, stresses vary with time either gradually or abruptly. Thus it is worth considering the condition in which a material creep under varying stress. There are three broad methods of computing creep under such circumstance. In all cases, we assume that within the range of stress-strength ratios

considered, creep is proportional to the applied stress, as such, one can therefore operate in terms of specific creep.

The first method of computing creep under variable stress is the effective modulus method. Here the elastic approach is applied but replace the modulus of elasticity by a sustained modulus based on the total deformation excluding that of shrinkage at a given time. Thus

$$E_t = \frac{1}{e_{sp} + C_{sp}} \qquad - (34)$$

where

C_{sp} = specific creep

e_{sp} = specific elastic deformation

E_t = sustained modulus of elasticity at time t.

Ross / 331 / however pointed out that the approach is theoretically unsound for varying stress because it disregards the stress history, since strain at any time being assumed to depend on the stress at that time only. Thus, the effective modulus also known as age - adjusted modulus method predicts complete recovery of strain on unloading, and this of course does not

really happen in practice. It follows that if stress decreases, the method under-estimates the strain

Bazant / 33 /, Jakov and Lazic / 33 / have worked on the effective modulus method. In his work, Bazant / 33 / assumed that the stress is within working stress range (below 40 per cent of ultimate strength) and the strain reversals are excluded. He also assumed that the creep of concrete is governed by the linear principle of super-position in time. That the elastic modulus also varies with time, and an unbounded final value of creep. After using the sum of squares to compare the former effective modulus equations with available data, he came up by improving the creep coefficient by introducing a parameter called aging coefficient. This idea was supported by Jakov and Lazic / 230 / who further improve the formula by generalising it to a form which they said is more accurate for composite beam structure in flexure.

The second method under variable stress is the rate of creep method which is based on the assumption that at any age of concrete, the rate of creep is independent of the age at loading. Thus, when the rate of specific

creep is $\partial c_{sp}/\partial T$ and the stress is σ, the increament in creep during an element of time ∂T is $\sigma \partial c_{sp}$, Thus, creep under a variable stress after a time t, since first loading is

$$C = \int_0^{t_1} \sigma \frac{\partial c_{sp}}{\partial t} \partial t \qquad (35)$$

$t = T - K$, $\partial t = \partial T$.

T = age of concrete

K = age at loading

This method takes into account to some extent the stress history but this apply only to increases in stress. Thus, when a stress is removed,

$\partial c_{sp}/\partial t \approx 0$ and no creep recovery results.

Furthermore, the rate of creep method assumes that $\partial c_{sp}/\partial t$ is independent of previous stress history. Thus under a decreasing stress, creep is over estimated and vice versa.

The rate of creep method is not any better than the effective modulus method since under a gradual decrease of stress both methods do not give the actual creep behaviour of concrete.

Only Bazant / 48 / has worked in this area of creep analysis{rate of creep method}in which he observe that at low stress, there is a significant, although ignored, non-linearity that consist in gradual stiffening or adaptation to a sustained compressive stress. It is of two kinds (1) an acceleration of the age-dependence of stiffness and (2) a stiffness increase due to compressive preload. The former is modelled by a stress-dependent equivalent hydration period and the later by an adaptation parameter whose rate is a function of the stress and the age.

The third method is based on the principle of superposition. Here each stress increment produces a resulting deformation component continuing for an infinite time. A stress decreament is considered as an increment with a negative sign. Thus creep-time curves are required for the various ages at loading at which an increament is applied. The effects of all the stress changes are summed, specific creep strains under compression and tension being assumed equal in magnitude. Gradual variation in stress can be treated as a series of small, finite increments, and summation can be applied.

Ross / 331 / tests on the three methods under variable stress showed that the method of superposition is superior to the rest two methods and that of rate of creep is better than that of effective modulus. One should also bear in mind that work by Bazant has also improved the effective modulus method recently. However, the effective modulus is simpler to apply.

Finally, one would like to point out under this sub-section that, more recently, a method called rate of flow was suggested by England and Illstons / 129 /. This is based on the division of creep into reversible and irreversible components. The method assumes the elastic strain to remain constant and the delayed elastic strain to follow an exponential curve. The irreversible creep is assumed to be due to viscous movement and therefore to be independent of stress history. The rate of deformation is proportional to stress and decrease with the age of concrete. Other researchers who work in this area are Bazant / 36 /, Screrbokov and Zaitsev / 345 /. One must stress here that rather than using a particular method under variable stress, the investigators used either one or two methods (for instance, rate of creep method and principle of superposition or effective modulus and principle of superposition)

in combination with one or two to three mechanisms of creep. As such, one end up with the same expression or equation with either additional parameters or an improvement of the expression determining the parameter through improving the quotient. Thus repeating the expressions is therefore not necessary.

2.5.0 RHEOLOGICAL MODELS

As earlier stated, the study of the relation between stresses and strains in the most general sense is termed rheology. Rheological models are theoretical ideal bodies (models) with strictly define rheological properties similar to the rheological behaviour of the real materials. In other words, the concrete time-deformation is simulated by imaginary rheological models consisting of elements, each of which represents a specific deformational characteristic of a given component or phase of concrete. This approach is however emperical and its success depends upon the ability to assign a specific part of the deformation of concrete to a given element of the model. Others combine a number of rheological elements in an attempt to simply approximate the observed overall deformational pattern without regard to their physical significance.

The basic models and the various other models proposed by researchers will therefore be reviewed in this section.

2.5.1 ELEMENTS OF RHEOLOGICAL MODEL

The idealised deformations which are used to build up real behaviour are elastic, viscous and plastic. They are represented by a spring, a dashpot and a friction element respectively. The bodies (models) with these ideal linear properties are referred to as Hookean solid, a Newtonian liquid and St. Venant body with respect to elastic, viscous and plastic. It should be emphasized here that these mechanical devices (models) do no more than represent the deformational behaviour observed in the real materials. It is not to be understood that there is any behavioural similarity between a rheological element and the mechanism of deformation of real material. The rheological models infact represent physical properties of continua and are therefore concerned with the macroscopic point of view only. They are therefore termed macro-rheology. Macro-rheology is entirely different from micro-rheology which is the study of the relations between the mechanical properties of multi-phase composite materials and the mechanical properties of the phases.

The approach here is through the use of geometrical models which resemble the true structure of the materials as closely as possible. Hence an understanding of the relation between structure and mechanical properties may be obtained. However, micro-rheological analysis may be used only when the phases involved can be regarded as continuous media in which cement gel does not satisfy, and this analysis cannot provide a key to the fundamental processes of deformation and failure.

In addition to the basic elements (spring, dashpot and, less frequently used, a friction element), some of the more elaborate rheological models also use other mechanical devices such as spring moving in dashpots with dashpot walls exerting friction to the spring, uni-directional dashpots, sorption elements, and sets of springs giving way under different load levels. The use of these additional elements recognises a departure from simple visco-elasticity for which springs and dashpots are sufficient.

2.5.2 BASIC MODEL.

The basic elements discussed above can be used to build up rheological models of varying complexity. However, only the basic models from

which complex concrete rheological models that were built will be discussed under this subsection.

There are two basic rheological models known as Kelvin (Voigt) and Maxwell models (bodies). This is presented in appendix A-41. In the Kelvin's model, the spring and dashpot are in parallel so that they undergo the same displacement and the total force on the model being the sum of the forces on the individual elements. It should be noted that in all rheological models, any horizontal arm (frame) is assumed to move parallel to itself. In Maxwell model, the spring and the dashpot are in series so that they take the same load, and the total displacement being the sum of displacements of the two elements. The direction of the force applied to the model is immaterial in both cases.

Considering the Kelvin's model, when load is applied suddenly, the elements exhibits no instantaneous deformation, but if the elements are linear, the deformation increases with time exponentially. At first, all the loads is carried by the dashpot but gradually transferred at a decreasing rate to the spring. At infinite time, the spring would carry the entire load. Thus, the deformation of Kelvin model approaches assymptotically a value equal to the instanteneous

deformation of the spring alone. In the model (Appendix A -41 , α denotes compliance of spring and V, denotes viscosity of dashpot. The applied force is represented by P, or σ for most of the models to be discussed. Appendix A -42 shows the deformation under a sustained load and after its removal. The full recovery is approached asymtotically. For this reason, Kelvin model represents the phenomenon of delayed elasticity and the behaviour is essentially elastic but with time-dependence due to the presence of a viscous body sharing the load. In the Maxwell model, when a load is applied, the deformation is proportional to the compliance of the spring and inversly proportional to the viscosity of the dashpot. Thus, high viscosity imply that the model will degenerate into a Hookean spring while on the other hand, low value of spring compliance (than means very stiff spring), the model will degenerate to a dashpot. Since its deformation is unlimited, the maxwell model essentially represent a liquid. Behaviour under load is also shown in appendix A-42.

The Kelvin and Maxwell models can be used to build up more complex but still simple basic models. The most basic built-up model is the Burgers model which is a series combination

of a Kelvin model and a Maxwell's model. This is shown in appendix A-43 with the deformational response of the model as a sum of responses of its Kelvin and Maxwell components. Thus under a sustained load, Burgers model, an instantaneous deformation take place, followed by a time-dependent deformation increasing at a decreasing rate and tending assymtotically to an inclined straight line on a deformation - time graph. On removal of load, an instantaneous recovery takes place, followed by a time-dependent recovery exponentially approaching a horizontal straight line. A part of the deformation under load is not recovered. Thus, the behaviour of a Burgers model is qunatitatively similar to that of concrete under creep.

2.5.3 MODELS FOR CONCRETE

For a qualititative simulation of rheological behaviour of concrete subjected to a sustained stress and then unload, Vaishnav and Kesler / 387 / consider a model for such concrete to have the following characteristics:

1) initial set on loading, partly constant, and partly stress-dependent,

2) instantaneous elastic deformation,

3) delay-elastic deformation,

4) a creep property dependent on stress and time

5) a permanent (irrecoverable time - dependent set, predominating at stress nearing the ultimate,

6) instantaneous elastic recovery,

7) delay-elastic recovery, and

8) Parmanent residual deformation.

When considering loads in the working stress range, conditions (1) and (2) can be combined and (5) ignored. Thus Vaishnav and Kesler / 387 / using the above conditions modified the Burgers model as presented in appendix A-44 by giving expression different from the normal viscosity of dashpot and spring compliance. The values are different for each element. This modification is to provide for rapid early creep or creep recovery. Due to the hydration of cement paste under load, the Maxwell dashpot should be taken to be of the age-thicknening type with infinite viscosity at infinite time. This may mean a finite creep depending on the law governing the age-dependence of the viscosity of the Maxwell dashpot. As it is generally believed, creep of concrete tends to be a finite limit, as such, the law of the Maxwell

dashpot should be such as to satisfy this requirement. Thus Kelvin model response to load governs creep at early stages and the Maxwell respose at later times. This model (the modified Burgers) they say is in good aggreement with creep results for concrete.

Over the years, various researchers have postulated various number of rheological models of varying complexity. These models are based mainly on the visco-elastic concept of concrete. Some of the models from various school of thoughts are presented. However, before showing the various models, it is worth presenting the idea of Jordan /221/ on viscoelastic models. He pointed out that concrete is a heterogeneous composite material composed of aggregate (coarse and fine) bounded together by hydrated cement paste. Thus he represent each material of the composite material by each element of the Maxwel, Kelvin and Burgers models. This is shown in appendix A-45.

Ross /327/ model was probably the first devised to apply to concrete. The model consists of a container with a porous piston with a spring above and below. The container in which the Piston moves is filled with a viscous fluid. The model is shown in A-46(a) It is similar to Kelvin with spring in series. Application of load produces instantaneous deformation on

the first top spring and if the load is sustained, a progressive deformation of the second spring take place (delayed-elastic). On removal of load there is an instantaneous recovery on the top first spring followed by a gradual recovery in the second spring (delay-elastic recovery). To simulate irrecoverable deformation, an opposing stress is assumed to act on the piston, during the reverse motion. Ross /327/ assumes shrinkage to be equivalent to an internal stress. This model is suitable as a first approximate to creep of concrete but does not provide for relaxation of stress. Fluege /136/ also developed rheological models based mainly on the experimental creep data of Davis and Davis /101/, and Glaville /158/. The model is presented in A-46 (b) also. It consist of a Kelvin model in series with two dashpot. One of the dashpot (A) has a constant viscosity and the other (B) a viscosity increasing with square of time. The dashpot of the Kelvin model has a coefficient of viscosity increasing linearly with time and the spring is also linear with compliance α_K. Under stress, the deformation of the dashpot B and of the Kelvin component ultimately attain finite values with time and (A) continues to increase indefinitely at a constant rate. The model is fitted only to a specific concrete tests. There is a wide variation in values of constants with the type

of concrete, storage conditions and age at loading.

Other rheological models include that of Hansen /175/ shown in A-47, in which the various elements are assumed to correspond to cement paste, voids and aggregate, correlating the physical components of the concrete with the parts of the models; (Jordan /221/ seems to have got the ideal from here). This model is substantially similar to that of Fluege. The Cowan /94/ model shown in A-48 which consist of a set of brittle springs (with graded rapture strengths) in series with two Kelvin models. One of the Kelvin model has a dashpot with a non-return valve. This results in non-recovery of some of the time deformation. The model was primarily devised to represent the stress-strain response of concrete loaded at a constant rate of strain and is of interest mainly in that respect.

Freudenthal and Roll /137/ postulated their rheological model in an attempt to recognise the various phenomena contributing, in their opinion to creep, so that components of their model are identified with possible mechanism of the several postulated components of creep. The model is shown in A-49 and consist of three Kelvin models and one Maxwel models, all in series. The Maxwell component of the model has a linear spring and a non-linear

dashpot with respect to force and time representing the long-term visco-elastic response in shear resulting in irrecoverable creep. The three Kelvin models in series has the first one with linear spring and dashpot representing the visco-elastic interaction in shear of the solid and fluid phases in concrete leading to a recoverable creep (delayed elasticity), the second is a non-linear Kelvin spring and dashpot with respect to the applied stress and represents the short-term consolidation effects due to irrecoverable seepage and lastiy the third has also a non-linear model and represents the irrecoverable deformation due to partially destructive internal re-adjustments within the concrete mass. The last two Kelvin components of model respond to increasing forces only, while the other two(Kelvin and Maxwell) react to both increasing and decreasing forces.

Powers /312/ however, critizes the use of dashpots in rheological models for concrete as a representation of viscosity and non-recoverability. Hence, he introduced a sorption element in place of dashpot. This sorption element may be in more accord with the physical phenomena actually involved in a recoverable creep which is his main interest. His model is shown in A-50 (a) and consist mainly of Kelvin model in which the dashpot is replaced with

a sorption element. The spring represents the elastic reaction of the non-evaporable part of the system while the sorption element represents two solid surfaces slightly separated by load-bearing water, whose thickness is up to 10 diameters of a water molecule. The sorption element differs from a dashpot in that it can maintain a static force and can account for recovery in the absence of a spring in the system.

An improvement of this model is that constructed by Gopalakrishnan et al /165/ shown also in A-50 (b), which applies to basic creep of neat cement paste only and consist of three elements in parallel. The meaning of the various terms used in the diagram are as follows:

V = the viscous deformation element;

M = the moisture diffusion element;

E = the elastic element of unhydrated cement.

The V-element has a spring S_1 to represent the elasticity of the gel skeleton in series with dash-pot D_1 which represent the rapture of existing links. The daspot has a non-return valve. The parallel dashpot D_2 represents the viscous deformation of adsorbed water under normal and shearing stresses.

The M-element contains two sorption cells and has the following sub-elements

C_1 = solid surfaces oriented at right angles to the applied load

C_2 = surfaces parallel to the load

S_3, S_4 = stiffness of the two springs and the ratio of the stiffness is equal the elastic poisson's ratio . This is because the stress in adsorbed water is proportional to the elastic strain.

The arrangement of C_1 and C_2, together with springs S_3 and S_4 is such that under load, the stress on C_1 and C_2 is in directions opposite to one another. The magnitude of the stress is controlled by S_3 and S_4. There is a connection between the two sorptions cells so that moisture diffusion can take place in order to bring about equilibrium of the free energy.

The E-element representing the deformation of unhydrated cement has the stiffness of spring S_5 which is inversly proportional to the degree of hydration.

On removal of load, creep recovery takes place but is smaller than the preceding creep owing to the non-return dashpot and to loss of water from the two sorption cells.

A-51 (a) shows the rheological model of Torroja and Paez /374/. They assumed that there is a limiting value of creep after an infinite time

under load and that there is an irrecoverable part of creep after infinitely long recovery. This is represented by a reserviour of viscous fluid connected to dashpots with springs so that liquid is drawn in or expelled from the cylinders containing the springs. The level of liquid in the reserviour is constant. The model also consists of several systems coupled together representing creep as being molecular in nature and essentially statistical phenomenon. The elements differ from one another in viscosity, frictional and elasticity coefficients but not in their intrinsic structure. The statistical distribution of strength within concrete is represented by an unequal frictional coefficients of the various rows of elements. It is claimed by Torroja and Paez that the model can describe deformation due to shrinkage and swelling, and creep. An increase in the ambient retative humidity represent an increase in the level of liquid in the reserviour and vice vasa. The inter-connection between the different pistons can represent the redistribution of stresses within the specimen due to creep and shrinkage. However, due to the absence of a free dashpot relaxation is not provided for by the model. The total extension of spring is defined by equating the external force to the sum of frictional resistances of the spirals. If the

applied force overcomes the frictional resistance of all spirals, failure of the element is deemed to have taken place. The simple form of the Torroja and Paez rheological model is also presented in A-51 (b) as an elementary model.

The Bjuggren's rheological model /65/ involves movement of water and also takes account of shrinkage as well as creep. This is shown in A-52. The elements A, B, and C represent respectively the elastic, plastic and viscous behaviour, while D represent shrinkage. The equilibruim position of the meniscus in the telescopic capillary tube of D is determined by the relative humidity within hydrated cement paste. The capillary force give rise to tension between the tubes, which increases with a decrease in the diameter of the meniscus. He assumed that both the applied load and the capillary force cause elastic, plastic and viscous deformations of the hydrated cement paste. Under compression, the meniscus rises and the shrinkage force decreases. As hygral equilibrium is disturbed by the application of an external force, water evaporates from the capillaries until equilibrium is re-established.

Glucklich /161/ proposed two rheological models. Both of the models are shown in A-53 and A-54 (a).

The first model consists of two Kelvin's models in series, connected in series to two springs with varying frictional restraint between them. One of the Kelvin's models contains a non-return valve. The slipping of the ends of the springs in between them represents an irreversible deformation on application of load. The creep deformation is due to the Kelvin models, one of which provide for reversible, and the other for irreversible creep. The second model consists of a set of three frictioned springs connected in series with a quasi- Kelvin model. The quasi-Kelvin model also consist another three frictioned springs in series. This implies that part of the creep deformation is reversible but the irreversible part of it is a function not only of stress but also of deformation.

Based on tests of concrete in torsion, Glucklich and Ishai / 162 / constructed a rheological model to represent the behaviour of concrete in torsion. This model and that proposed by Ishai / 207 / alone are shown in A-54 (b) and A-55. That of Glucklich and Ishai A-54(b) is mainly Maxwell model in series with a spring of the same rigidity to that in Maxwell, all in parallel with a Kelvin model having a spring

instead of dashpot with both springs having the same rigidity. The rigidity of the Maxwell model spring is different from that of Kelvin. The Maxwell dashpot contains partially empty pores to represent voids in concrete.

That of Isahi / 207 / model represents a quantitative description of visco-elastic creep in torsion. Ishai believed strongly that creep of concrete is due to water migration in concrete of three orders of magnitudes. Thus his model consist of three Kelvin models and a linear spring, all in series. In the model the various mechanism is explained below. The model is shown in A-55

Mechanism 1 - Zeolitic water migration (inter-layer)

Mechanism 2 - gel water migration

Mechanism 3 - Capillary water migration.

He believes that residual creep is caused by the zeolitic water migration.

CHAPTER THREE

EXPERIMENTAL TESTS

3.1.0 ANALYSIS OF LATERITIC MATERIALS

The fact that lateritic materials vary in their texture, compositions and occurrence especially those of the tropics as earlier noted cannot be overemphasized. Thus, their performance vary greatly. In order to understand the lateritic materials used in this thesis, some of the geotechnical properties of the materials, its location, the geology of the area and the quality of the material were investigated. The analysis of such results may lead to the understanding of the behaviour of the soil-crete materials. In other words, the rheological behaviour of soilcrete materials could be as a result of the constituent materials making up soilcrete - (constitutive laws). The various analyses carried out are presented below.

3.1.1 LOCATION AND SELECTION OF MATERIALS

The lateritic material used in the thesis was obtained from beside the blocks of nine flats in Area F Residential quarters of Ahmadu Bello University Zaria main Campus. This is situated in the area of peneplain (primary) laterite in the Zaria area. Lateritic deposits is widely encounted all over Nigeria. In northern part of Nigeria, the plains on which Zaria is situated are part of the vast, gently undulating

plain scenery which extends almost unbroken from Sokoto in Norther-West to Lake Chad in the North-east and beyond, from south of Kaduna to the Tiqueddi Scarp near Agades and surmounted by hill features of rock inselbergs and lateritic ironstone. Previous studies of lateritic materials in the Zaria area /290/indicate that the lateritic materials are developed in situ from different parent materials with or without the introduction of externally derived iron. The lateritic materials containing chiefly the concretionary dark-brown type hard and porous on exposed surfaces, grading downwards through yellowish brown ochreous and mottle lateritic materials into deeply weathered kaolinized and ironstained zone over-lying the bed rock. Observations around the location of the lateritic samples showed quartz lenses and veins running more or less undisturbed from the granite parent rock into the laterite above.

Geologically, Nigeria contains mainly the Younger Granites with radiometric ages which vary from carboniferous in the north to cretaceous in the south, and occupy about 80 per cent of the total surface area of the country. The Younger Granite is a type of rock which is mainly a high level discordant intrussions into the basement

complex. Other type of geologic rocks are the Younger metasediment which consist mainly of pelitic and semi-pelitic schists, phyllites and subordinate amount of psammitic calcareous and meta-igneous rock; older granite suite which consist mainly of granitic to dioritic rocks; volcanics and hypabyssal rocks which consists mainly small groups of rocks intermediate to acidic in composition; and the charnokitic rocks which are mainly of intermediate to felsic composition with subordinate amounts of basic composition.

The percentage of the younger granite outcrops (hills/mountains) to the total area of the country is greater than for any other group of rocks. The younger granite consist predominantly of granites which can be subdivided on the basis of varietal minerals. This include Biotite granites, Rhyolites, Riebeckite granites, Amphibole-fayalite granites and other basic and intermediate rocks.

The volcanics of cretaceous to recent age occupy about 1 per cent of the total surface area of Nigeria. They are restricted, in occurrence, to the easter part of the country. Volcanism during cretaceous times appears to have been largely restricted to the Benue trough, while

volcanism of lower cenozoic age appears to be restricted to areas around the Jos Plateau. These volcanic rocks are predominantly basalts with smaller occurrence of more alkaline rocks like trachytes and phonolites.

In selecting this particular lateritic materials for soilcrete, various preliminary test were carried out on two type of lateritic samples observed in Zaria. The first type is grey in colour and found close to banks of rivers, streams and stagnant waters while other type which is concretionary dark-brown in colour grading down-wards through yellowish ochreous and mottled lateritic materials are found almost everywhere as the top soil. In the preliminary test, soil-crete blocks are formed using each of the laterite and its wet compressive strength determine the suitability of each lateritic materials. It was observed that while concretionary dark-brown lateritic materials undergoes strength development when soaked in water, the greyed coloured one disintergrated undergoing reaction, with the soil-crete block pealing off while soaked. This could be as a result of high silt content or high organic matter and swelling element present in the sample. The concretionary dark-brown lateritic materials was

thus employed in soilcrete production and much of the physical and physico-chemical tests were performed on this type of lateritic material which is actually more common in Zaria and all over Nigeria.

3.1.2 DETERMINATION OF PHYSICAL PROPERTIES

Soil generally may be taken to include any loose sedmentary deposit. In other words, it could include sand, clay, gravel, marl and so on. In order to be able to discuss the properties of different types of soil, it is necessary to have some ways of classifying them. There are several ways in which soil or lateritic materials may be classified. Such ways include geological origin, by mineral content, by grain size or by plasticity. The last two are widely used by soil Engineers and the determination of consistency limits and other physical properties which will enable one classify the lateritic material is the subject of this subsection.

The various physical tests carried out on the lateritic materials in the soil mechanic laboratory of the Civil Engineering Department, Ahmadu Bello University, Zaria is summerised below in Table 6.

TABLE 6 PHYSICAL PROPERTIES OF THE LATERITIC MATERIAL

Serial Number	Physical Properties Tested	The value obtained
1	Moisture content (%) as tested in January, 1988	0.54
2	Liquid Limit (%)	41
3	Plastic Limit (%)	26
4	Plastic Index (%)	15
5	Group Index	5.4
6	Linear Shrinkage (%)	10
7	Specific Gravity	2.45
8	Maximum dry density kg/m^3	1760
9	Optimal Moisture content (%)	14

The distribution of the various-sized particles in the lateritic materials is shown in the Particle-size distribution curve of Figure 1. One observe that the lateritic material can be said to be a well graded sand according to the particle-size distribution chart.

3.1.3 DETERMINATION OF PHYSICO-CHEMICAL PROPERTIES

Physico-chemical test is the determination by physical means the chemical composition of a substance with the determination involving chemical reaction or etching to reveal metallic structure.

Lateritic materials can seldom be termed pure, although in some, one mineral type may predominate while the impurities present may be also be all-important in determining its properties. Thus, it contains many mineral types and are extremely variable in composition. However, it is worth noting that in their natural state, lateritic materials consists of mainly of minerals of primary origin and minerals of secondary origin. The minerals of the primary origin are those present in the parent rock from which the material have been formed and have undergone no significant alteration in composition while those of secondary origin is produced by the action of chemical and

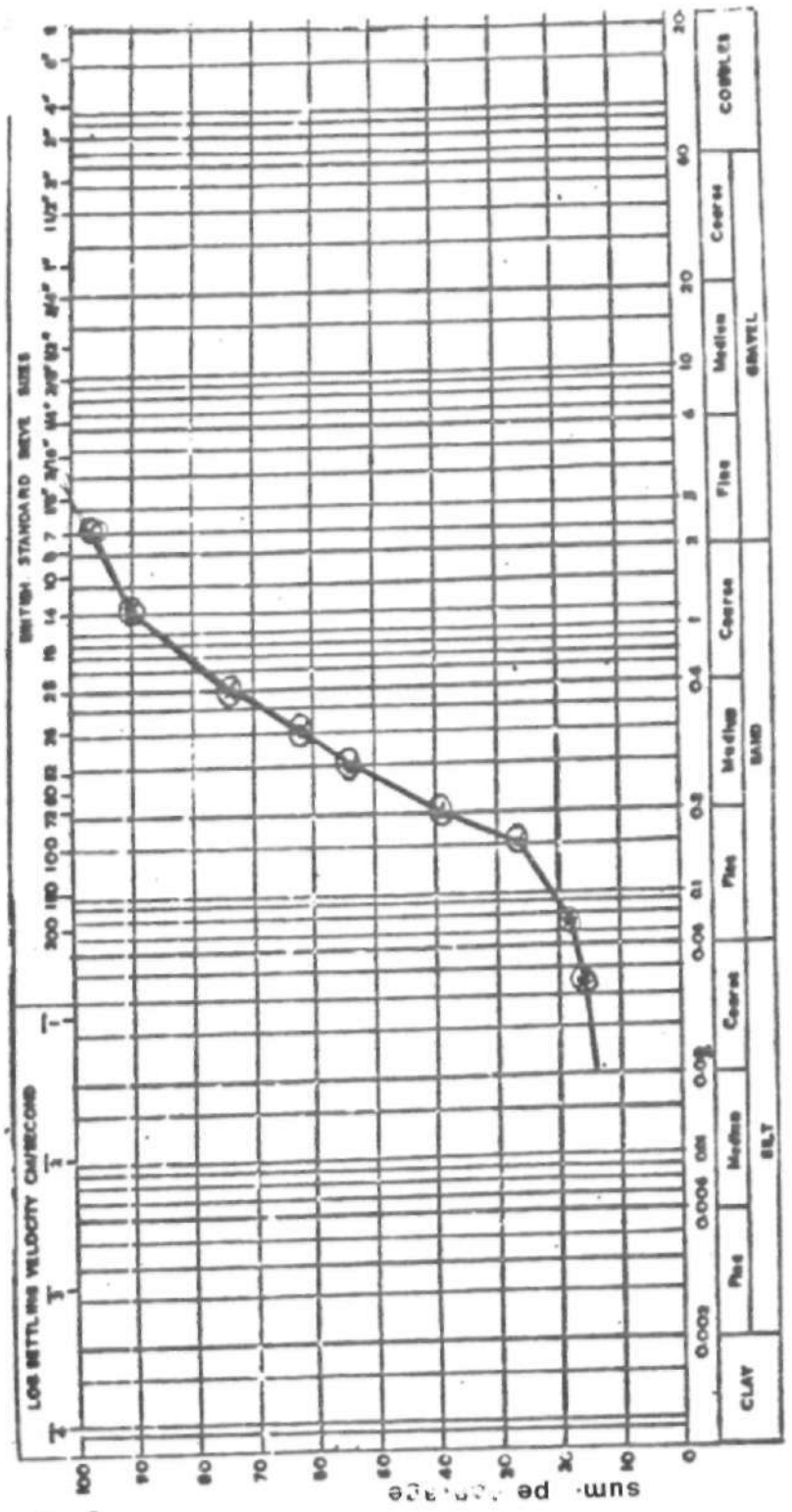


FIGURE 1. LATERITIC PARTICLE SIZE DISTRIBUTON

physical agents on primary minerals resulting in their breakdown and conversion to secondary minerals.

In order to know the chemical compositions of the lateritic material used in this thesis, some physico-chemical tests were performed on the material. For comparison purposes, physico-chemical tests were also carried out on the soil-crete materials. The tests were carried out in the Chemistry Department of the FMPA in Stuttgart, West Germany. The results are presented in Tables 7 and 8 below. The chemical compositions of both the lateritic and soilcrete materials were carried out using X-ray fluorescence analysis.

TABLE 7 ELEMENTAL COMPOSITIONS

Element/ Physical properties	Laterite Per centage	Soilcrete Content (%) by weight
1 PH	7.5	11.3
2 Moisture content	1.2	1.5
3 So3	0.10	0.20
4 cL^-	0.003	0.004
5 Co_2	0.16	0.93
6 Organic Carbon	0.18	0.06
7 Residium (HcL)	83.8	74.1
8 Residium (HcL + Na_2Co_3)	80.6	60.4
9 Loss of ignition (at 1000^oc)	6.2	10.0

TABLE 8 TOTAL ANALYSIS ON BASIS OF OXIDE CONTENT

Serial Number	Elemental Oxide Present	Laterite Percentage	Soilcrete content (%) by weight
1	S_iO_2	55.0	62.5
2	Al_2O_3	33.1	22.0
3	Fe_2O_3	9.2	7.2
4	C_aO	0.3	6.7
5	M_gO	0.1	0.2
6	K_2O	0.6	0.5
7	Na_2O	-	-
8	TiO_2	0.9	0.7
9	MnO	0.03	0.04
10	P_2O_5	0.05	0.05

The analysis of So3, cL^-, Co_2, residium by treating with hydrochloric acid, residium by treating with hydrochloric acid and sodium carbonate; and the loss of ignition at 1000°c; were according to DIN. 1164 /116/. The moisture content was obtained by drying at 105°c and the PH - value by suspension with water and 10% solid material.

3.1.4 X-RAY DIFFRACTION ANALYSIS

In the X-ray diffraction analysis by the Powder method, the lateritic and the soilcrete Powder samples are

each placed in turn in a beam of X-rays which is diffracted through a limited number of angles by the crystal latice of the sample. X-rays consist of eletromagnetic waves, similar to light but of shorter wavelength (in this particular test 1.541Å). They are produced when a fast-moving streams of eletrons impinge on a metal surfaced sometimes called anti-cathode. Thus, they result from the energy liberated when the electrons of the metal atoms are excited out of their normal orbits by the bombarding particles (electrons).

The atomic arrangement of the powdered materials (powdered method) placed in the X-ray beams are determined by X-ray diffraction and provides a means to measure the spacing of the atomic planes (sheet thickness). The distances thus measured are expressed usually in angstroms (10^{-10}m). The X-ray diffraction patterns are usually recorded on a photographic film as a series of lines in the form of arcs concentric with the original X-ray beam. From the position of these lines which is determined by the angle of defraction, and hence the separation of spacing of the planes of atoms producing them, can be deduced. These sets of atomic spacings are characteristic of the minerals producing them. In the case of the lateritic materials or clay

minerals, the highest spacing observed is the most important for identification purposes. It is this distance between the sheets of atoms that are characteristic of the clay minerals and is known as basal spacing.

However, in this particular X-ray powdered deffractogram, more accurate and sophiscated method was employed in determing the minerals in both the lateritic as well as soilcrete materials for comparison. The tests were carried out by the same Chemistry Department of the FMPA, in University of Stuttgart, West Germany.

As earlier explained, the identification of minerals by X-ray is based upon the atomic spacings which is characteristic of the deffracted line pattern. The intensity of the reflections depends upon the arrangement of atoms in the crystal, the number of reflecting planes and the intensity of the X-ray beam, When the radiation are controlled, the intensity of lines, for a certain beam, will be in proportion to the number of reflecting planes, or, in other words to the amount of that mineral present in the sample. Thus for quantitative and accurate method of X-ray analysis, the total intensity during an exposure at a particular angle is measured. This is achieved by using the Geiger-

Mueller Counter which is a type of electrostatic voltmeter used for measuring the intensity of X-ray radiations. Many x-ray cammeras are designed to incoperate this counter. As each reflected ray (diffracted ray) is encountered, the intensity of the beam is recorded directly on a micro-volmeter and the positions of the reflection and their absolute intensity are determined in one operation. The intensity measured by the Geiger-Mueller counter is, in the simplest terms, the x-ray flux falling on the instrument per unit time. A particular reflection which is characterisitic of the mineral to be estimated, is allowed to fall on the counter, and the resulting deflection on the voltmeter is measured. The counter is then adjusted to measure the intensity of the original direct beam or a small fraction of it if desired. Thus, in this particular test, the deffrracted x-ray beam intensity is measured rather than the spacing giving the unit in Angstroms. As such, the units in the ordinate axis are in counts per second (CPS) in the preceeding x-ray diagrams.

A powered sample is an aggregation of fine crystallites, usually with random orientation. An x-ray beam striking such a sample will be

reflected by all planes of atoms which are present in the minerals in the powder. Because of this the sample placed in the x-ray beams are rotated at various angles and the intensity measured at each angle. Thus the units of the abssica are in degrees in all the preceeding x-ray diffractograms.

In this particular x-ray diffraction tests, an x-ray radiation beam known as monochromatic radiation of a fixed wavelength (copper $K\alpha$, λ = 1.541 Å) is focused on the powdered samples. This monochromatic radiation with Nickel as filter is used because a high intensity can be focused on the sample without serious deterioration of the sample.

The sample is then rotated at various angles and the corresponding intensity measured. The x-ray diffraction diagrams characterisitic of the mineral compounds present in the lateritic sample, soilcrete sample lateritic-soilcrete sample and the compounds present in the lateritic sample are shown in figures 2, 3, 4, and 5 respectively. One can see that quartz and kaolinite are predominant in the laterite while in the soilcrete there is more quartz but less kaolinite.

3.1.5 CLASSIFICATION OF LATERITIC MATERIAL

Using the results obtained from the physical and phsico-chemical tests, the lateritic material is

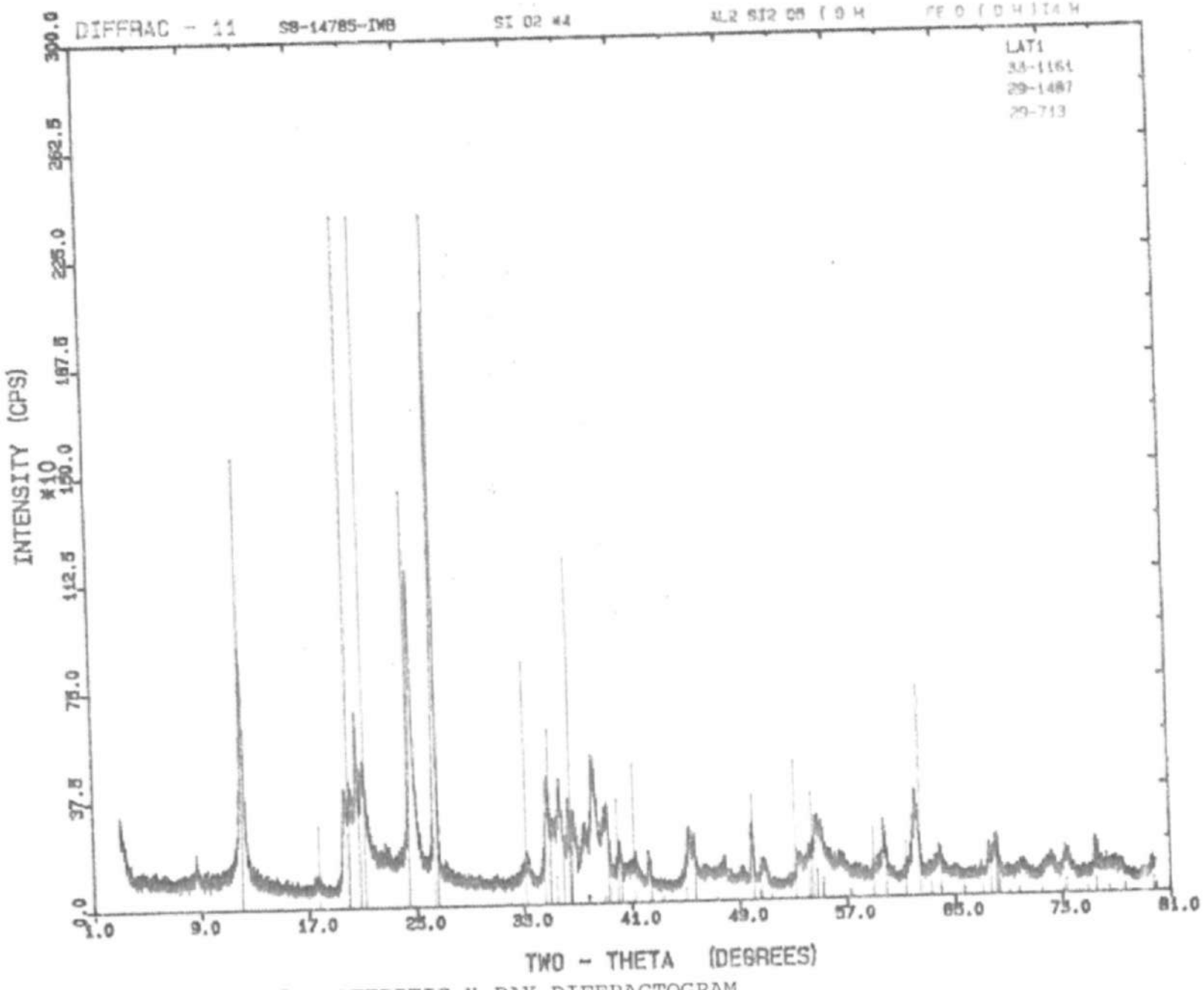


FIG.2. LATERITIC X-RAY DIFFRACTOGRAM.

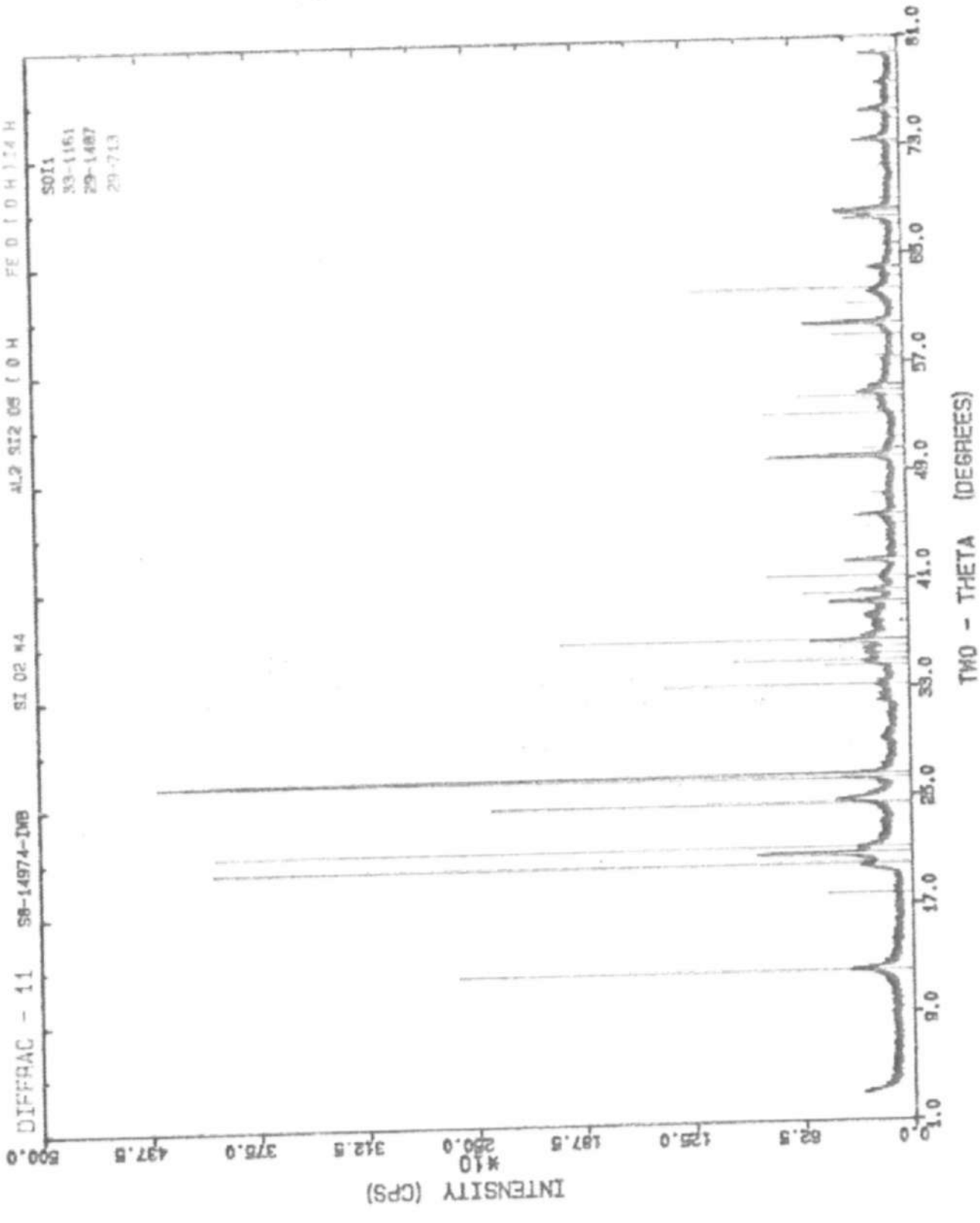


FIG.3. SOILCRETE X-RAY DIFFRACTOGRAM.

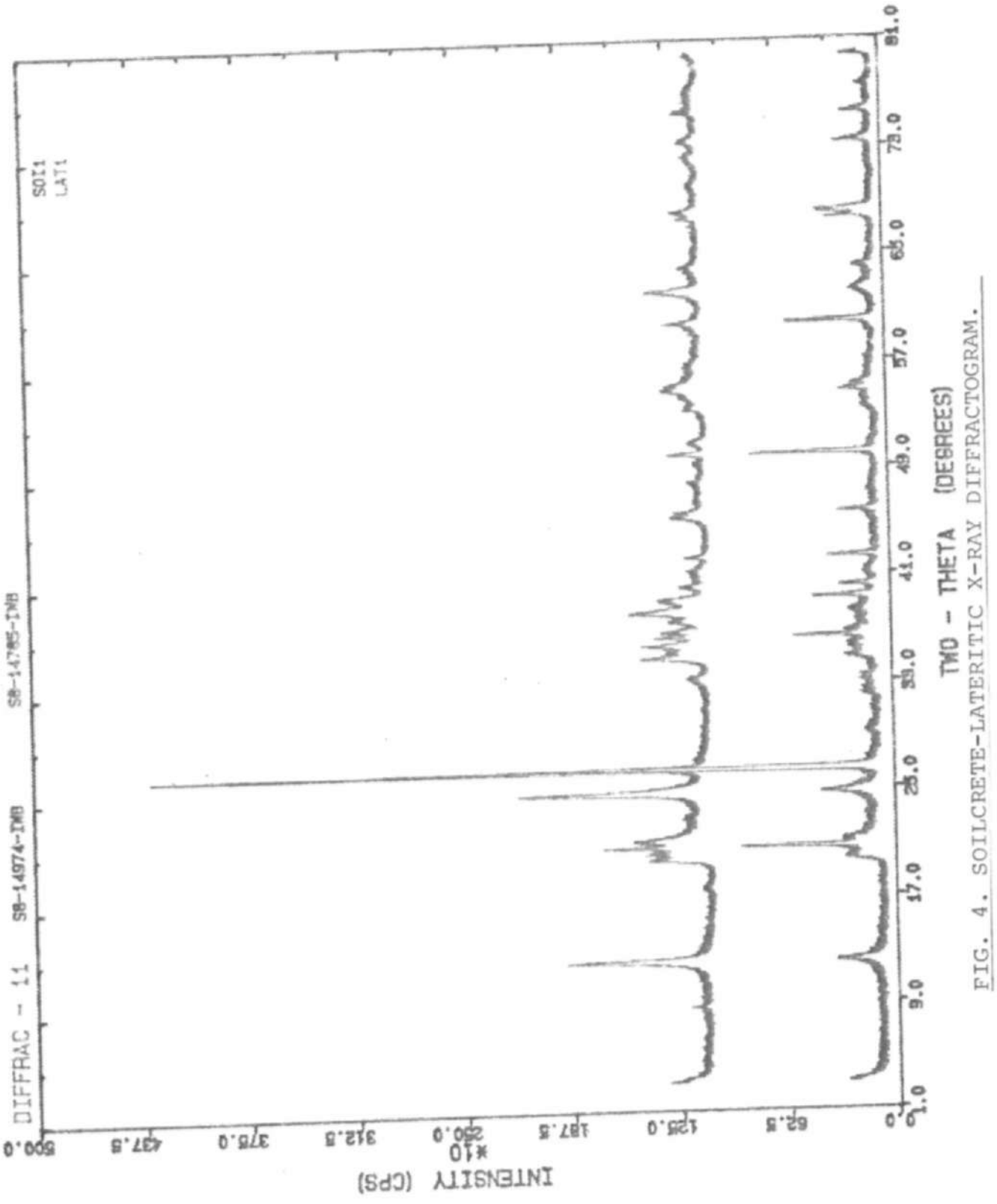
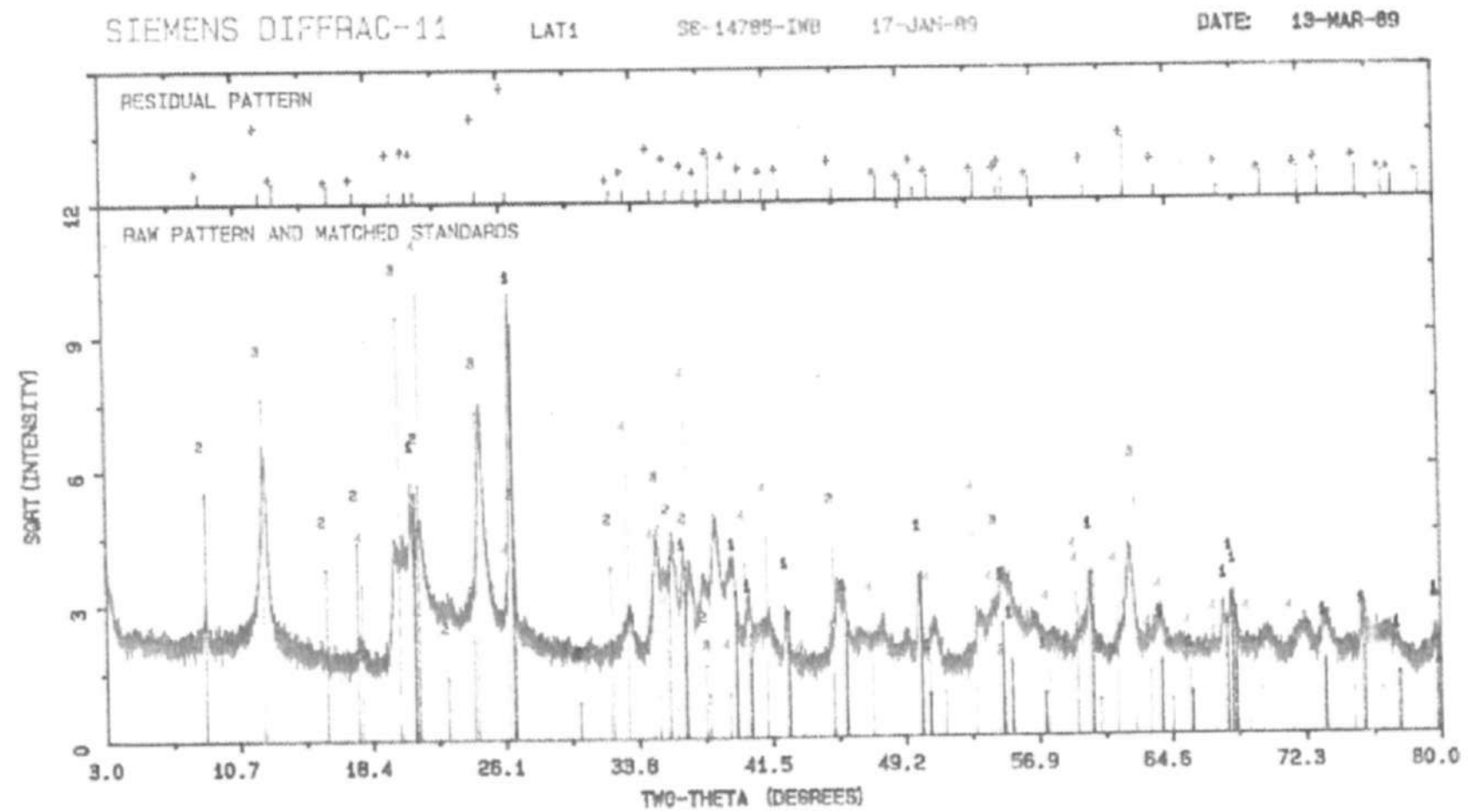


FIG. 4. SOILCRETE-LATERITIC X-RAY DIFFRACTOGRAM.



	FRACT	STANDARD		I/ICOR	FOM
1	9	5.0490	SIO2	3.6	0.631
2	13	16.0365	CU4AL2(CO3,SO4)(	1.0	0.088
3	36	29.1487	AL2SI2O5(OH)4	1.0	0.100
4	40	29.0713	FEO(OH)	1.0	*****

FIG.5.LATERITIC COMPOUNDS X-RAY DIFFRACTOGRAM.

classified as shown below. However, it is worth noting once again that the classification methods for temperate lateritic materials which is based on their mechanical behaviour do not give satisfactory results with the tropical lateritic materials unless additional factors like alumina-iron ratio and silica-sesquioxide ratio are invoked. Thus this particular lateritic material is classified using the various methods proposed by various researchers.

(a) United States Bureau of Public Road Classification

Using the values of liquid limit, plastic limit, plasticity index and the group index obtained in the physical test; and the percentage passing number 36; and 200 British Standard Sieves which is 62 and 19 per cent respectively, the lateritic material is classified according to the above method as A-6 (Plastic Clay Soil).

(b) According to the Particle-size chart as seen in Figure 1, the material is classified as well graded sand of intermediate plasticity.

(c) Casagrande's Classification

This system has been developed for roads and airfields. The soil type is usually designated by two capital letters. Using this method of classification, the material is classified in the major

division as Fine grained soil having medium plasticity with the sub-group classification as Inorganic clays of medium plasticity (CI).

(d) Using the modified Konta /153/ classification the material is classified as lateritic clayey residuum as shown below in Figure 6 (No. 2).

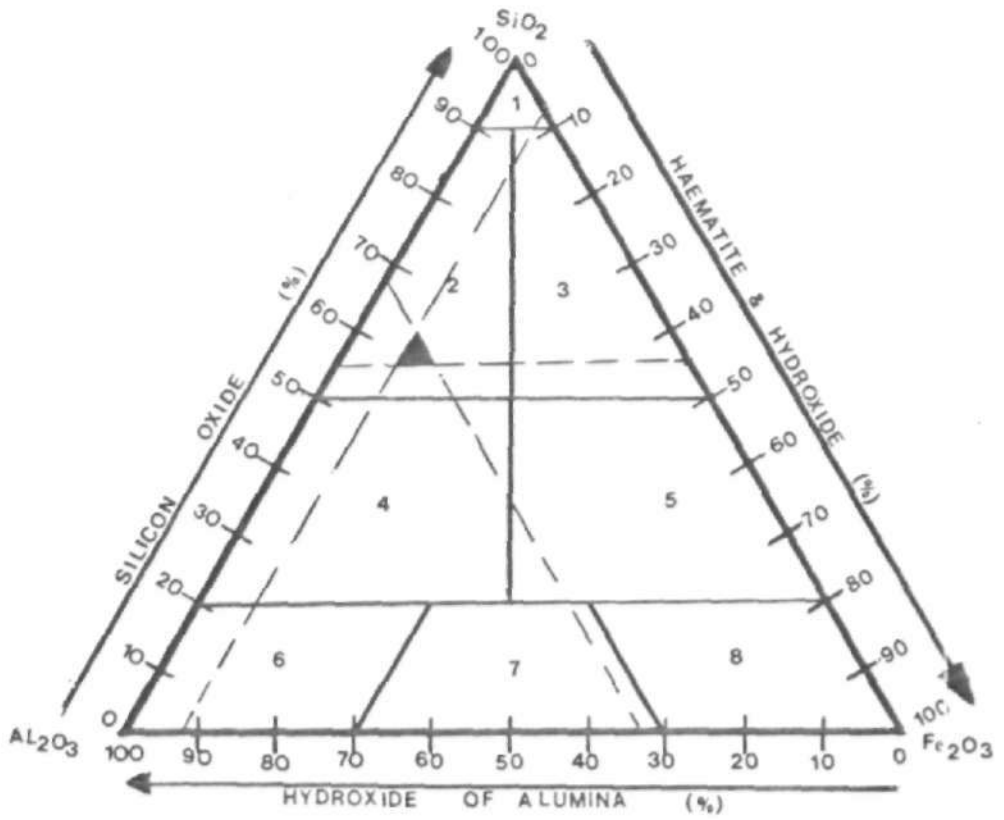


Fig. 6 Modified Konto Classifiaction

(e) Using Gidigasu /145, 147, 152/ it is classified as Modular or concretionary laterite. Applying the process of laterisation, the same lateritic materials can be classified by Gidigasu as laterite and lateritic quartz gravels.

(f) Howerver D'Hoore /117/ would have classified the same material as Ferruginous laterite since he based his classification on annual rainfall and; liquid and plasticity limits.

(g) On the other hand, Ajayi/ 7 /would have said it is a Coarse lateritic materials using grain size distribution.

(h) Although Nanda/268/ would classified it as Residual Laterite because he believed that latertitic material should be classified according to the way they are formed. In this case, it is formed in-situ under free drainage condition.

(i) Even though Ola/290/ would have classified the material as Laterites because the silica to sequioxide ratio is less than 1.33, in this case 1.30, he also believes that it is not convenient especially where there is no adequate laboratory facilities. As such, he thinks the use of atterberg limits for the usual classification is sufficient.

One believes that any of the above method of classifications can be used depending on the situation at hand. Where one can perform laboratory tests and there are adequate facilities, the use of atterberg limits, grain size distribution chart and silica sequioxide ratio can be employed. Other-wise grain

size, annual rainfall if known, and the drainage of the area can be used as a quick guide to classify the material.

3.1.6 OBSERVATIONS

From the test results obtained above, one can observe some important relation which are discussed below.

(a) Specific Gravity

The average specific gravity as determined using standard laboratory method at a temperature of $65^{o}c$ is 2.46. The specific-gravity values for soil particles are a useful parameter needed for all calculation involving void ratio, porosity and the degree of saturation. Gidigasu/153/ proposed that in laterite soils , the particles generally have a specific gravity between 2.5 and 4.6. An average decomposed quartz-rich parent materials without a high degree of laterization have a specific gravity between 2.5 and 3.0. This is because quartz which forms almost half or more of many clays is the main constituent of most sands which has a specific gravity of 2.65 while that of the common primary clay minerals is within or not too far outside this value. The high value of specific gravity in lateritic soils are however generally associated with soils rich in iron oxides and titaniferous minerals.

Thus the value of 2.46 obtained is correct as the lateritic sample has a quartz-rich content of oxide of silica which is 55 per cent and iron oxide of 9 per cent combined with little amount of Titanous oxide of 0.9 per cent. The low value of iron oxide (less than 10 per cent) tend to reduce the value of specific graveity while the high content of silica tend to raise the value to about 2.5.

(b) Colour

Lateritic soils have characteristic redish coulour which appear to be due to the various degree of iron, titanium and manganese oxide present in the soil. The colour also reflect the degree of maturity with aged ferruginous lateritic soils changing from red to brown to black while aluminous soils become brighter in colour. Generally natural soils derive their colour from two sources.

1. From organic matter which is usually black, brown or grey
2. From mineral composition with
   (i) Iron giving red, orange, yellow, brown, blue and green depending on what type of iron.
   (ii) Calcium, magnesium, sodium and potassium

giving white colour

(iii) Aluminium also giving white or grey colour and

(iv) Mangasese giving black and brown colour

From the chemical analysis, one observe that the material contains geothite $(Fe(oH)_3$ and hematite (Fe_2O_3) (Iron oxide - 9%) which gives yellow to brown colour; very small quantity of calcium (CaO - 0.3%) which gives white colour; high amount of aluminum $(Al_2O_3$ - 33%) which also gives white and very minute amount of manganese which gives brown colour. Thus, no wonder therefore that the colour of the tested lateritic material was light-yellow brown.

(c) *Silica Content*

The chemical test reveal that the lateritic material is rich in quartz with silica dioxide content of 55 per cent by weight. This quartz occurs mostly in the form of free silica and oxide of silica, with its presense being mainly in crystalline form.

This indicate that the parent rock from which the lateritic material is formed is mainly igneous rock, called granite. The effect of free silica in soil is that it lowers the plasticity and lessen

the shrinkage on drying. Thus, the lateritic materials have plasticity index of 15 instead of 20, or 26. It also indicate that the lateritic material is of insitu weathering.

(d) Physico-chemical Properties

As noted above, the primary mineral in the lateritic material is quartz while the secondary mineral include Kaolinite ($Al_2(oH)_2\ si_2O_5$), haematite (Fe_2O_3) and geothite ($Fe(oH)_3 \xrightarrow{-H_2O} FeO(oH)$).

However, the percentage of Kaolin present in this material (33 per cent), suggest that it is to high to be considered as a secondary mineral in the lateritic material. Thus, the granitic igneous rock from which the primary mineral, in this case quartz, was formed, also undergoes a breakdown under the action of natural agencies (weathering) given rise to minerals called felspars and micas but felspar is predominant as there was no trace of sodium in the lateritic material. In the presense of weak acids like carbonic acids and boric acids, the felspar changes to Kaolin-type of mineral. Secondly, the small quantity of mica present in the form of monoclinic plate-like masses or particles consisting of alumino-silicates of potassium, iron and magnesium give rise to products like muscovite mica which

yield hydrous mica and, finnaly back to the same Kaolin mineral. In addition to the above formation of Kaolin, it can also be formed under hypogenic action. That means a sub-surface reaction at high temperature and pressure (chemical weathering) and direct sediments of these Kaolins. Thus the actual secondary mineral are the haematite (Fe_2O_3) and the geothite ($Fe(OH)_3 - H_2O \rightarrow FeO(OH)$) which are less than 10 per cent.

Some minerals in soils can absorb cations (or bases) from solutions present in the soil. Thus, the lateritic material contains some cations or bases like calcium, magnesium, potassium, titanium, mangenese, all in small minute quantities. Of the cations present, calcium predominate with a percentage of 0.3. This implies that there is no swelling clay mineral like montmorillonite in the sample but purely Kaolinite which gives the lateritic material a strong support for engineering application. The type of cations which are present in natural soils though are associated with the mineral crystallites, also depends principally on the mode of formation and the environment. Thus, this lateritic material having calcium as the predominant cation suggest that the material is associated with lime strata and has not been

extensively leached since there is no hydrogen ions.. The rest cation or bases present are less than the amount of calcium ion and far less than one percent as shown in Tables 7 and 8 indicating that the material is safe from corrosion attack in presense of water.

Some other analysis carried out and shown in Tables 7 and 8 are worth mentioning as this affect the choice of the lateritic material for engineering purposes.

The organic carbon also known as organic or carbonaceous matter measured from the chemical analysis of the materials indicate very small quantity of 0.18 per cent. The lateritic material derive the mater chiefly from percolation of water containing humid acids and matter in solution or suspension, the organic matter being absorbed by the pores of the lateritic clay mineral thereby retaining them. The low content of the organic carbon, far less than 1% indicate the goodness of the material for stabilisation.

The loss on ignition at 1000^{o}c measured is 6.2 per cent. The loss on ignition at 1000^{o}c is regarded as an important property of any soil. Apart from the routine chemical analysis of materials, the total weight loss on calcining is

also used as a means of identifying some minerals and as a rough means of estimating the percentage of some fraction clay minerals (mainly Kaolinite). In this particular case it is used for the later. Thus in a soil in which the clay mineral is almost entirely Kaolinitic, that is to say that it is pure clay mineral, the weight loss or loss on ignition at 1000^{o}c is about 13.9 per cent. Thus, in this particular lateritic material with 6.2 per cent weight loss on ignition implies that 45 per cent of the mineral content is of clay mainly kaolinite.

The moisture content determined is 1.2 per cent at a temperature of drying of 105^{o}c. The moisture content also known as the loss of moisture which is the loss in weight after drying at 105^{o}c can be an important and an informative determination. This is because some clay minerals like montmorillonites which is a troublesome clay mineral can take up large quantities of moisture up to 20 per cent when exposed to the atmosphere, while clay minerals like kaolinite which is a good clay material hardly or rarely holds more than 2 per cent.

The materials being lost resulting into the loss of weight include the hygroscopic and adsorbed water, colloidal water, water of hydration and absorbed water which are removed at 105°c classified as the loss of moisture or moisture content; and the combined water which is part of the structure of the minerals, carbon dioxide or monoxide from the burning of carbonaceous material, sulphur dioxide and trioxide from sulphur compounds and loss of various other gases and volatile substances present in the material when burnt or exposed at 1000°c as loss of ignition.

The treatment with Hydrochloric acid and sodium carbonate is to check how the lateritic material can withstand the attack of acid and alkaline as these are also dangerous to concrete materials. It is an attempt to foresee whether or not structural units formed from such lateritic material can be used for stores or similar purposes for acid and alkaline substances. Though many acids are known to dissolve certain substances in soils and alter their chemical compositions as well as their physical form; and that Hydrochloric acid attacks many calcareous and ferruginous substances, the treatment with hydrochloric acid gives about 84 per cent of the laterite as residuum. Secondly,

the treatment with both hydrochloric acid and sodium carbonate solution at the same time give rise to about 81 per cent of the lateritic material present as residuum. This implies that the material can withstand acid and alkaline attack to some extend. It has also a good resistance to corrosion.

The value of the PH detemined is 7.5. The available data on the PH of lateritic material and non-lateritic material indicate that lateritic weathering medium is characterised by a neutral to acidic medium while tropical black clay medium is an alkaline reaction. The value of 7.5 can be said to be neutral which is once again emphasizing the fact that the lateritic material is of insitu weathing medium.

(e) Soilcrete Physico-chemical Properties

The chemical analysis of soilcrete give rise to an increase in oxide of silicon and calcium and a decrease in the oxide of aluminium and iron and the minor oxides virtually remaining the same. The silicon oxide had an increase of 7.5 per cent and that of calcium oxide is 6.4 per cent. The aluminium oxide is reduced by 11 per cent and that of iron oxide by 2 per cent. This increase and reduction in the various oxide can be explained as follow:

The increase in silcon oxide and calcium oxide are as a result of compounds present in Portland cement used in stabilising the lateritic material. The presence of tricalcium silicate ($3CaO.siO_2$) and dicalcium silicate ($2CaO.siO_2$) in Portland cement must have increased lime and silica content of the soilcrete. The reduction in aliminium oxide and haematite can be attributed to the formation of tricalcium aluminate ($3CaO.Al_2O_3$) and tetracalcium aluminoferrite ($4CaO.Al_2O_3.Fe_2O_3$) with increased lime content from the cement while mixing with water. The tricalcium aluminate and tetracalcium aluminate are also present in Portland cement. The presence of gypsum ($CaSo4.2H_2O$) also contributes to the increase in lime content and sulphur oxide ($So3$) which was increased by 0.1 per cent. The reaction of tricalcium aluminate with water is voilent (flash set) and the presence of gypsum slows it down and eventually giving rise to a new compound tricalcium aluminate hydrate (hydration product). The tetracalcium aluminoferrite also react with gypsum to form calcium supphoferrite and calcium sulphoaluminate and their presense accelerate the hydration of the silicates. The tri-and di-calcium silicates are of course the main compounds of hydration. The hydration product cannot be traced

by any physico-chemical anyalysis, as such there is only loss in the compounds responsible for the hydration.

The treatment with hydrachloric acid gives a residue of about 74 per cent. Both treatment with hydrochloric acid and sodium carbonate solution gives a residue of 60 per cent, almost half of the weight. The explanation of attack of hydrochloric acid and alkaline solution on concrete can also be said to be responsible here. The increase in loss of ignition is as a result of fineness of Portland cement and that of moisture content as due to some impurities in portland cement.

(f) X-ray Diffraction Analysis

In the x-ray diffractive analysis of the lateritic sample, the following mineralogical compounds were identified: quartz composed of mainly free silica and oxide of silica, aluminious clay mineral known as kaolinite ($Al_2(oH)_4 Si_2O_5$), iron hydroxide and oxide (geothite $-Fe(OH)_3 \xrightarrow{-H_2O} FeO(OH)$; haematite - Fe_2O_3), possibly a copper-bearing complex mineral and graphitic carbon. From Figure 2, the various elemental compound are represent by various colours. The highest peaks are shown by the

colours representing quartz, kaolinite, Geothite and haemetite. Figure 5 shows the x-ray diffractogram giving the various proportion of the mentioned compounds in the lateritic sample. The copper-bearing complex mineral can either exist as copper aluminium/sulphate-carbonate or, designated as $Cu_4Al_2(Co_3.SO_4)$. Since the principal impurities in clay soils are the complex silicates or alumino-sililcates containing other elements as principal ingredient, this copper bearing alumino-carbonate or sulphate can be regarded as impurities in the lateritic soil. These complex mineral could be an unaltered primary form of mineral or as a result of contamination with the true primary mineral. The presence of graphitic carbon could be as a result of metamorphism of carbonaceous material (organic matter) possibly sedimentary origin, since the quantity in the material is small. Thus, one cannot say that the area is a coal-bearing region where there has been a volcanic intrusion.

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Figure 3 and 4 shows the diffractegrams for soilcrete and soilcrete-laterite respectively. As earlier noted, there is more quartz and less kaolinite and iron minerals with the possible reasons given in (d) and (e).

However, one could notice a very high peaks in the diffractograms of figures 3 and 4 for soilcrete.

In figure 4 particularly where the lateritic and soilcrete diffractograms are put on the same scale, the soilcrete has an intensity of about 430 counts per second while the lateritic materials is about 300 counts per second. It is well known that hydration products of cement and water cannot be identified by the existing techniques like the combination of the scanning electron microscope and energy dispersive x-ray spectrometer. Though studies indicate that the hydration product exist in variety of forms. This include fibrous particles, flattened particles and reticular network of particles, all rather difficult to define. However, the predominant form is that of fibrous particle which is said to be possibly solid, or possibly hallow and somtime flattened. Since the sturcture of particles of hydration cannot be established by any technique because of its fineness of phases (particle size too small) and its disorderliness, the researchers seems not to have a clear picture of the structure. However, the process of hyration of cement can be determined by different means such as the measurement of

1. the amount of $Ca(OH)_2$ (lime in water)in the paste;
2. the heat evolved by hydration;

3. the specific gravity of the paste;
4. the amount of chemically combined water;
5. the amount of unhydrated cement present and
6. indirectly from the strength of the hydrated paste.

Thus the high peak noticed in the soilcrete could be the mineral of the released lime seperating our as $Ca(oH)_2$ after the hydration product has been formed or it could be the unhydrated cement paste and quartz from the laterite being indicated.

(g) Mechanism of Stabilisation

The soilcrete material is a mixture of cement, laterite and water. It is mainly 1 part of cement to 12 part of laterite by volume as compared to sandcrete blocks which is 1 part of cement to 6 or 8 of sand. The amount of water required is that sufficient to bind the materials together which is about 7.5 per cent. Thus it is a stabilised earth.

The gain in compressive strength of soilcrete material is as a result of the development of chains or agglomerations of hydrated cement and soils grains which enclose voids in a three-dimensional skeleton. Even though the gain in strength could take place without the lateritic materials, it

has been well established that the mechanism of cementitious action is in part a surface phenomenon and the high specific area of the lateritic clay minerals contribute significantly to the development of strength in soil-cement stabilization. Thus all fraction sizes as well as lateritic clay minerals appear to take part in the development of a new structure during the strength gain. This is represented in the sketch below as Figure 7.

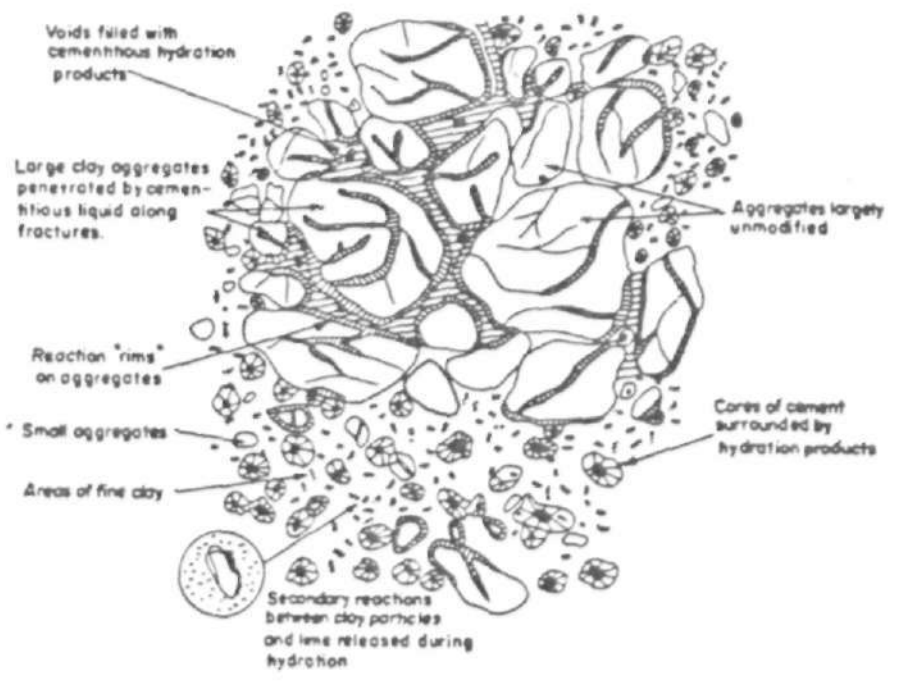


Figure 7 SCHEMATIC REPRESENTATION OF SOIL-CEMENT STABILIZATION.

3.2.0 COMPRESSIVE STRENGTH OF SOILCRETE HOLLOW BLOCKS

The main reason behind the various tests being performed on soilcrete materials is because soilcrete is being used as a structural units. The need for the construction of low cost houses preferrably from indigeneous and highly abundant material in any country cannot be overemphasized if the economy of such country is to be stable. Thus, a research toward the production of building units from local materials to alliviate the problem of housing accommodations faced by millions of people in Nigeria is in order.

Presently, soilcrete is being used to form hollow blocks measuring 150 x 400 millimetres in plan cross-section with a height of 150mm, and two hollows of size 155 x 95mm each. The physical properties of such building units as related to its compressive strength and as determined is presented in this section of this thesis.

3.2.1 PRODUCTION OF BLOCK SAMPLES

All the samples used in this section of the thesis were produced by ocidental construction company (OCC) Zaria, Nigeria. The main material in the production is the Kakeyi Lateritic Deposit which is also A-6-plastic clay soil as obtained from index properties of three sample taken from different

locations of the production site. This conform the statement of the lateritic material being found all over Nigeria as top soil. During production, the soil is dug vertically and shovelled several metres to obtain an even mixture. It is then sieved through a 19.05 millimetre cylindrical sieve and mixed with the determined amount of cement (usually about 8 per cent laterite) prior to mixing with water (about 7.5 per cent by weight of lateritic sample). The blocks are formed using TECMOR block making machine (made from Brazil) by means of hydraulic press. Four blocks are produced at a time but the compaction pressure under which the blocks are formed cannot be indicated by the machine. In other words, the machine cannot measure the pressure applied as such the exact value is not known, though it is assumed to be about 5 N/mm^2. Thus, the need to check the /-assumed opti-mum compaction pressure cannot be over-emphasized. The daily production of ocidental construction company is about three thousand blocks with a batch production of about 45 blocks. The commercially produced blocks are cured by spraying with water two times a day (morning and evening) and covering with a water-proof materials for at least seven days. Samples are then taken at random to laboratory for

test after 28 days of age or at the age required for test.

3.2.2 EFFECT OF CONTACT LAYERS ON COMPRESSIVE STRENGTH

It was observed that the materials used between the bed faces of the soilcrete blocks and the platens of the testing machine during the compressive strength test has a considerable effect on the strength of the soilcrete hollow blocks. In other words, the packing materils used between the block and the testing machine result to various compressive strength values. This may be due to the fact that distribution of load uniformly to the bed faces varies with the materials in contact between the block bed faces and the platens.

For bricks and sandcrete blocks, 3-ply plywood, plaster of paris and sand-cement of 1:1 are normally used for standard test. For soilcrete, various contact materials (packing materials) between the block and the platens of the testing machine are used in order to determine the best packing material to be used. The packing materials used include:

(a) Sand-cement motar (1:1) of not less than 6mm thick - usually 6 - 10mm thick.

(b) 3-ply plywood sheet with each ply 3mm thick

(c) Soft board (Celotex) of 10mm thick

(d) soil-cement (1:8) of not less than 6mm thick

(e) Steel platens - with no packing materials.

To ensure that this property is characteristic of soilcrete materials, various other soilcrete mixes were carried out and the strength after 28 days following the commercial curing using the various packing materials is as shown in Table 9. The normal soilcrete is 8% cement content (1:12).

Table 9 COMPRESSIVE STRENGTH OF SOILCRETE BLOCKS USING DIFFEENT PACKING

Serial No.	Surface contact Packing material	Compressive Strength (N/mm^2)				
		8%	10%	12%	Aver-age	% of Av. strength
						Assumed
1	Softboards (celo-tex	2.82	2.58	3.75	3.05	100
2	Plywood	2.78	2.28	2.67	2.58	85
3	Steel platens	2.40	1.76	3.17	2.44	80
4	Soil-cement	2.22	2.0	2.88	2.37	78
5	Sand-cement(mortar)	1.98	1.75	2.71	2.15	70
6	Softboard (celo-tex	2.05	2.14	2.16	2.12	100
7	Plywood	1.53	1.54	1.50	1.52	72
8	Steel platens	1.44	1.16	1.87	1.49	70
9	Soil-cement	1.91	1.65	2.01	1.86	88
10	Sand-cement (mortar)	1.80	1.41	1.75	1.65	77

Serial number 1 to 5 implies dry strength

Serial number 6 to 10 implies wet strength

Sample tested are selected at random and tested in wet and dry state. The test values shown are an average of four samples.

3.2.3 DEVELOPMENT OF DRY COMPRESSIVE STRENGTH

To utilise any building units, the idea of when the dry strength is fully developed is of importance. Thus tests were carried out on the compressive strength of soilcrete blocks at various ages to check on the development of the strength. This is shown in figure 8 with the values shown on the figure as the average value of four samples tested between softboards.

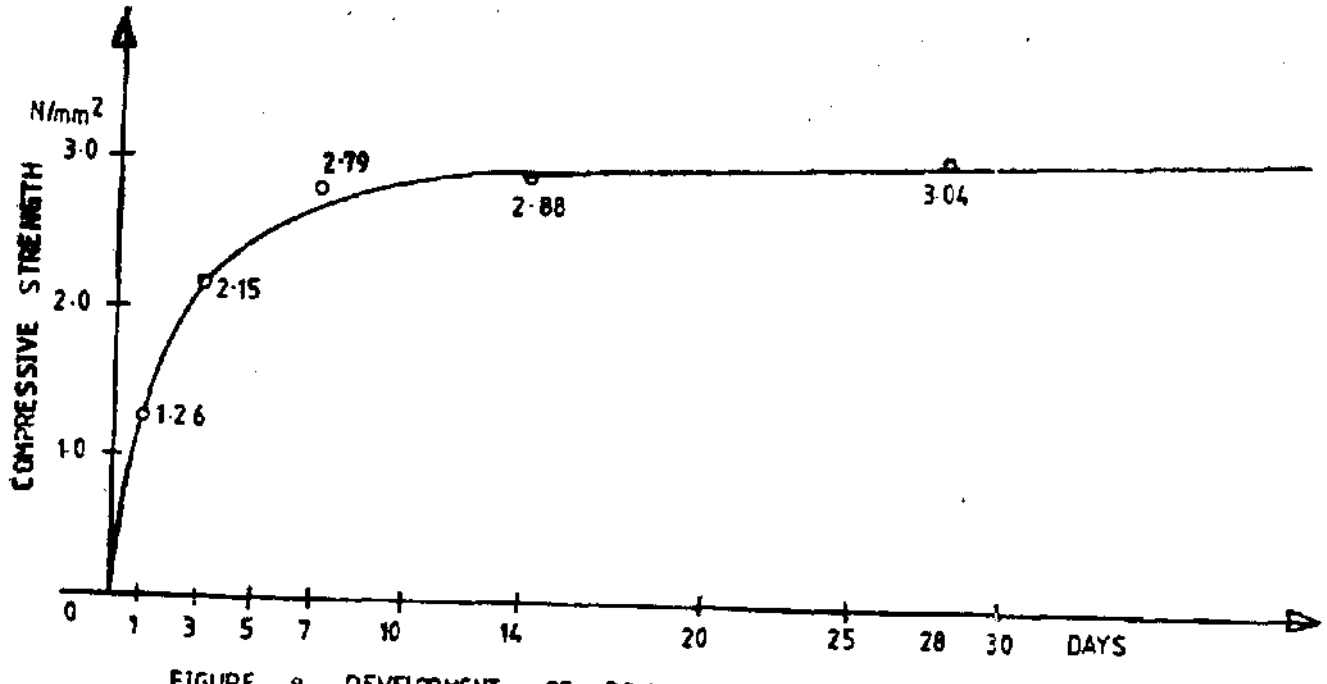


FIGURE 8 DEVELOPMENT OF DRY STRENGTH

Commercial curing were employed for both dry and wet development of compressive strength. Test on the development of dry compressive strength were carried out for longer period up to two years and the result is as shown in Figure 9.

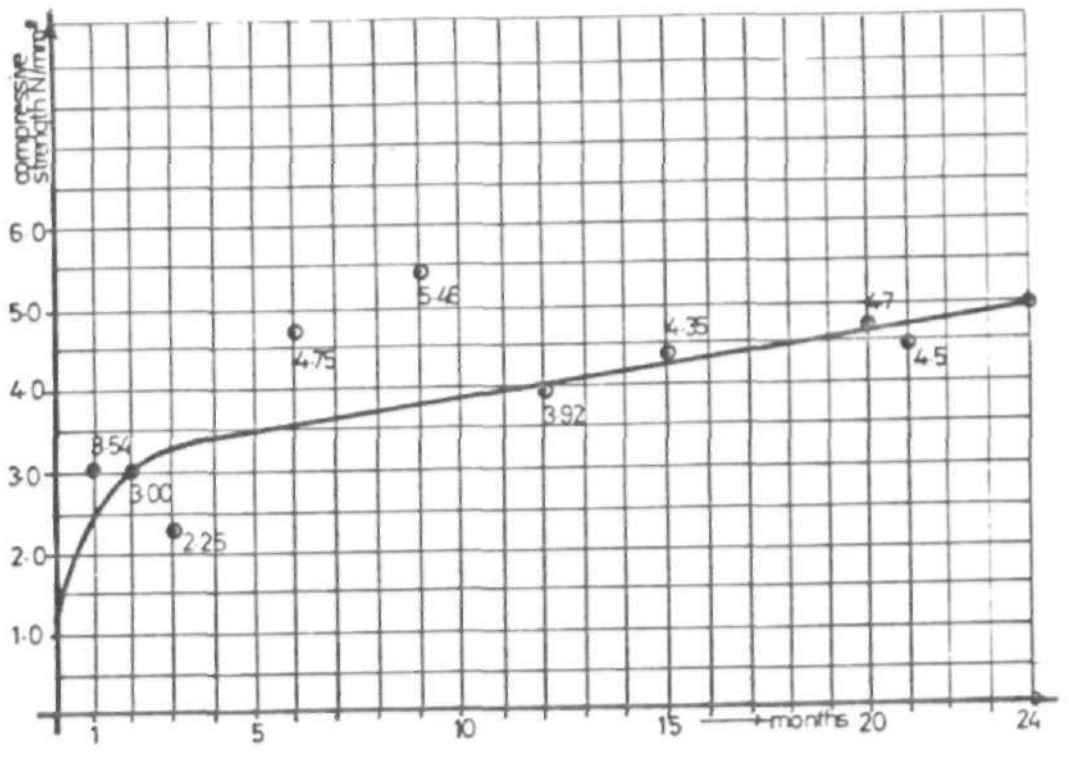


Figure 9 Development of Dry Strength Over 24 - Months

3.2.4 DEVELOPMENT OF WET COMPRESSIVE STRENGTH

Structural building units usually have lower strength when tested wet than when tested in the dry state. It therefore present the fact that wet

compressive strength test represents the worst conditions in practice and are therefore the safer to use. It has been considered to be the better tests to apply for general purposes. For sand-crete and clay bricks, the soaking periods ranges from 1 to 8 days in water prior to compressive strength tests for standard purposes. After 28 days of age, the randomly selected soilcrete samples are soaked completely in water and the compressive strength at various soaking ages were determined. The results is as shown in Figure 10. Compressive stregnth were determined using soft-boards as packing materials.

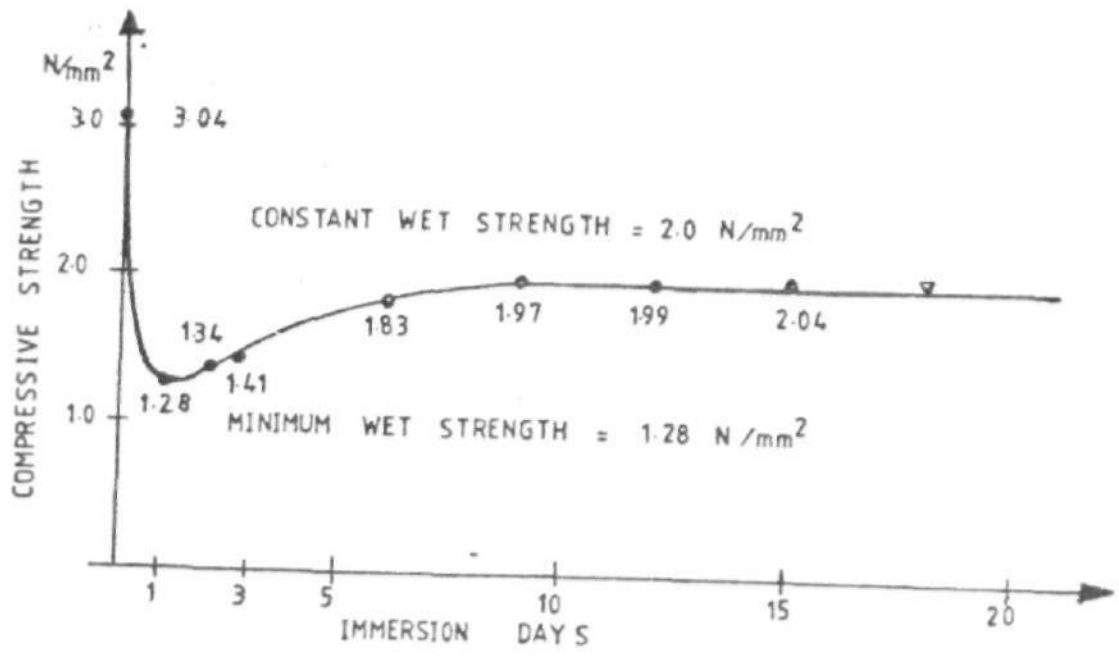


FIGURE 10 WET COMPRESSIVE STRENGTH

3.2.5 METHOD OF CURING FOR SOILCRETE

Some cracks were observed in some of the soilcrete samples produced using the commercial curing. The need therefore came up for tests of better way of curing. Though some of the cracks are as a result of shrinkage, it was also noted that the need to check the method of curing cannot be overemphasized.

The first test on curing was carried out on the site of production. Immediately after production, three randomly selected samples were completely soaked in water. It was observed that it crumbled after 10 to 15 minutes in water.

The second test involves the following:
About 36 randomly selected block samples produced from one batch were completely soaked for 1, 3, and 7 days in water after an age of 6, 12, 24 and 48 hours. For each age, about 9 samples were soaked with 3-samples each for 1, 3 and 7 days. After soaking for the required days, the blocks are then removed and left in the laboratory atmosphere until it is 20 days of age. It is then resoaked for 1 day and tested after 21 days of age for wet strength. The blocks were tested with sofboards as packing materials. The results are as shown in Table 10.

TABLE 10 MINIMUM WET COMPRESSIVE STRENGTH OF SOIL-CRETE BLOCKS AFTER DIFFERENT METHOD OF CURING

Number of days soaked before drying → (days)	1	3	7	Average percentage increase in 1-day soaking strength (1.28 N/mm^2)(%)
Age of blocks before soaking (hours) ↓	Compressive strength after 1-day soaking in water (N/mm^2)			
6	2.00	2.04	1.59	46
12	1.39	1.61	1.57	19
24	1.73	2.19	2.51	67
48	1.93	2.06	2.21	61

Those soaked after 24 and 48 hours of production have an average strength increase 67 and 61 per cent respectively of the minimum wet compressive strength (1.28 N/mm^2). Those soaked after 12 and 6 hours have generally low strength increase.

3.2.6 OBSERVATIONS

The results of table 9 clearly show that the softboards (celotex) used as a packing material gave higher compressive strength than the rest packing materials. Thus, the celotex in the case of soil-crete blocks will distribute the load uniformly over the block's bed faces. Assuming the celotex values obtained to be 100 per cent, the compressive

strength of solicrete blocks is underestimated by 20 per cent when plywood and soil-cement mortar are used as packing materials and 25 per cent when steel platens and cement mortar are used as packing materials. In both wet and dry compressive strength, the reduction in using each packing material is virtually the same.

In the development of dry compressive strength, the blocks developed about 90 per cent of their 28-day strength within one week of age as shown in Figure 8. However, the strength continues to increase with age as a result of hydration as shown in Figure 9 with the curve pointing gradually virtically at an inclined direction. Concrete also show the same behaviour.

In the wet strength development (figure 10), the dry strength drops to about 40 per cent of the 28-day dry strenth after 1 to 2 days of soaking. Then there is a re-development of strength which comes to a steady wet compressive strength as from 8 to 15 days soaking period prior to testing. Thus the critical soaking period is that which gives the minimum wet strength which is about 1 to 2 days. This time of 1 to 2 days before the re-development of strength may be due to the insufficient moisture which is as a result of time required for the mechanism of stabilisation in presence of moisture to take place. This behaviour is also observed in creep

of concrete /236/.

Taking the ratios of wet compressive strengths at various soaking ages to that of 28-day dry compressive strengths, a graph as shown in Figure 11 is obatained.

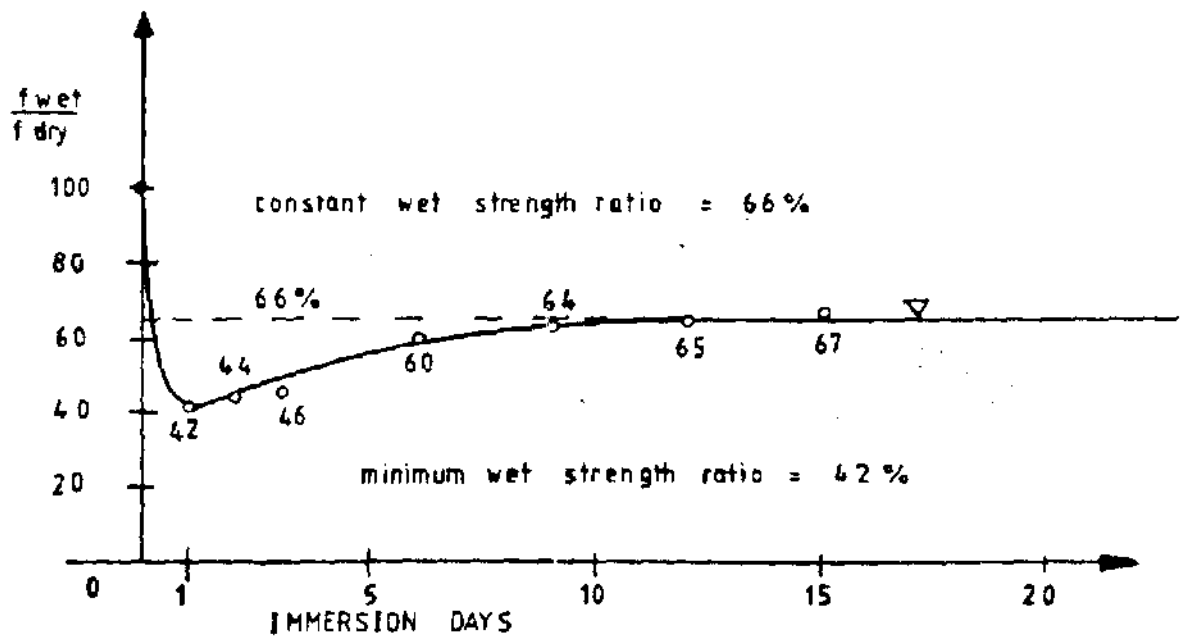


FIGURE 11 f wet/ f dry RATIO IN PER CENTAGE

One can observe that the steady ratio is about 65 per cent of the 28-days dry compressive strength.

Since the soilcrete blocks has its minimum wet compressive strength after 1 day soaking in water and in the test carried out to determine the best method of curing, all the compressive strength are

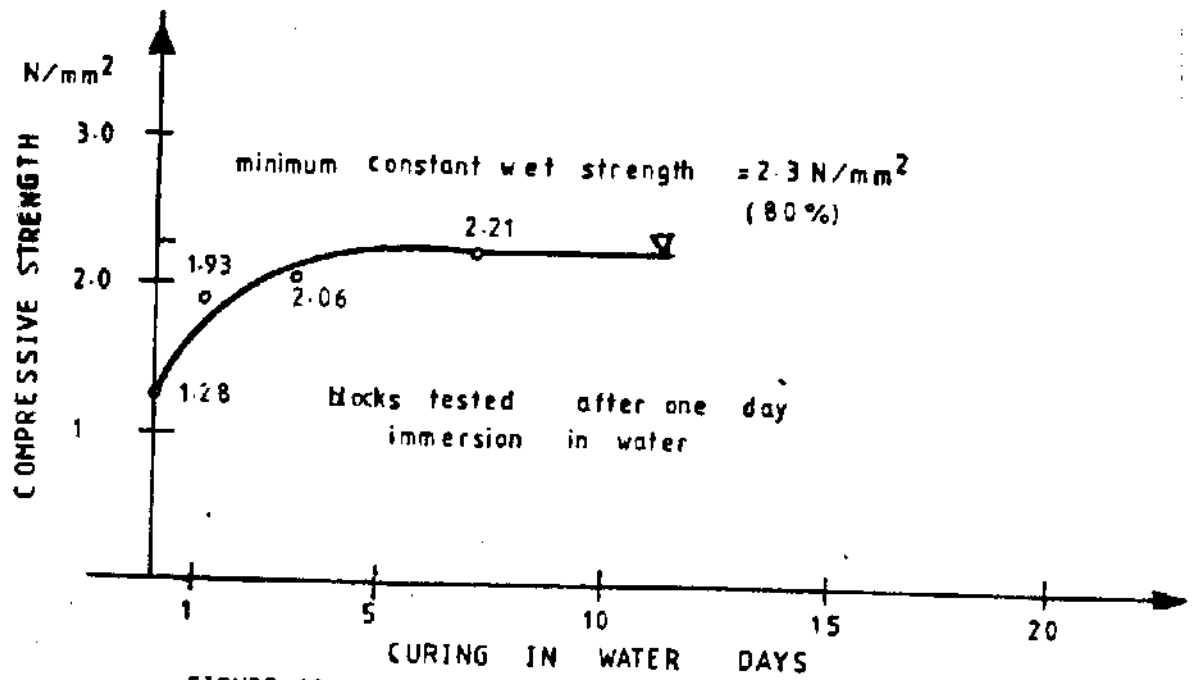


FIGURE 13 WET COMPRESSIVE STRENGTH OF 48 HOURS AGE BEFORE CURING

One observe that the minimum wet compressive strength without this type of curing is 1.28 N/mm^2 which is now taken as zero type of curing. Its value has now been increased to 2.55 N/mm^2 and 2.30 N/mm^2 called minimum constant wet strength. Thus after 24 or 48 hours of age before soaking in water as a method of curing, the minimum constant wet strength was increased by 100 per cent and 80 per cent respectively. One also observe that soaking for 2 days after the two ages almost give the minimum constant wet strength especially figure 13.

Thus the best curing method for soilcrete is after 24 hours of age, it is soaked for 48 hours (2-day) and left to cure in the laboratory atmosphere for 28-days before testing.

For commercial purposes, it would be unwise and uneconomical to use soaking in water method of curing since the samples would be in thousands and require extra space and container for such purposes. However, for standards and research purposes, the method of soaking could be employed. Whatever method of curing is employed, one day soaking prior to testing is best for wet compressive strength.

For other various tests carried out on the soil-crete in the preceeding section of the thesis, the method of soaking for 48 hours after an age of 24 hours and allowed to cure in laboratory atmosphere was adopted.

3.3.0 SHORT-TERM DEFORMATION TESTS

As earlier explained short term deformation is mainly concerned with modulus of elasticity in compression in the case of this thesis. The tests, materials, results and its analysis of the deformation are as presented below.

3.3.1 MATERIALS FOR EXPERIMENTS

The materials for the test is mainly soilcrete samples of cylindrical shape. The production and other physical properties essential for the production of the cylindrical samples as determined were as presented below.

(a) Water - Aggregate Ratio

In order to determine the most suitable water content for production of soilcrete cylindrical specimens, various tests were carried out as follows. All the lateritic sample used in the test are those passing through sieve size of 4.76mm. Before the laterite were stabilized with cement, the moisture content were determined and the result is as presented in Table 11.

TABLE 11 MOISTURE CONTENT OF LATERITIC SAMPLE

Bottle No	32	66	51
Bottle + wet laterite (gm)	28.20	28.10	28.20
Bottle + dry laterite (gm)	28.10	28.00	28.10
Bottle empty (gm)	9.40	9.70	9.80
Dry weight of laterite(gm)	18.70	18.30	18.30
Weight loss (gm)	0.10	0.10	0.10
Moisture content (%)	0.53	0.55	0.55

Average moisture content = 0.54%

added until the samples are mouldable. At this time, the water-cement ratio is about 1.7 by weight. It was at this ratio that cylindrical specimen were mouldable. Thus it appears that the concept of water-cement ratio is not very suitable for soilcrete materials. The laterite which is the main bulk material in this case absorbs a lot of water unlike sand, as such basing the water content on the amount of cement is unrealistic.

Another test was then embarked upon using the concept of water-aggregate ratio instead of water-cement ratio. The aggregate in tnis case is purely lateritic materials. The result is as presented below.

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TABLE 13 WATER -AGGREGATE RATIO TEST

Water-Aggregate Ratio	Compaction pressure N/mm^2	Remarks
0.05	7.2	Cylindrical specimens were produced but with difficulty. All specimens produced are with cracks. Specimens Soaked after 24-hours crumbled under water
0.06	7.2	
0.08	7.2	
0.09	7.2	
0.10	7.2	
0.13	7	Produced without cracks but crumbled under water
0.17	7.2	Produced without cracks but crumbled under water.

0.20	7.2	Prôduced without cracks and did not crumbled under water
0.30	4.8	There is excess water at this ratio as water started coming out at a pressure of 4.8 N/mm^2. The mix was extruding out of the steel mould at both top and bottom

One observe that the water content improves as one goes down the water-aggregate ratio. At the onset, most of the specimens were produced with heavy cracks up to a water-aggregate ratio of 0.10. It was only as from water-aggregate ratio of 0,130 that samples were produced without cracks, l t however, crumbled in water immediately after soaking at the age of twenty four hours. At water-aggregate ratio of 0.200, the specimens were produced without cracks and did not crumbled under water after soaking at the age of 24 hours. Normally, the best method of curing for soil-crete is by soaking in water for 48 hours after an age of 24 hours as described in section 3.2.5. At the water-aggregate ratio of 0.3, the water becomes excess, such that at a compaction pressure of 4.8 N/mm^2, the mix was extruding out of the steel mould at both top and bottom. At this point further detail test were carried on the following close water-aggregate

ratio to determine exactly the best ratio. Such water-aggregate ratio are 0.190, 0.200 and 0.201. It was observed that at water-aggregate ratio of 0.201, further addition of water may lead to too much water as water is seen coming out at the base plate after compaction pressure of 7.2 N/mm^2. The results of the tests on the three water-aggregate ratios are as presented below.

Table 14 COMPRESSIVE STRENGTH RESULTS

Water-aggregate ratio = 0.190

Age of cylindrical specimen at test = 28 days

Compaction pressure used = 7.2 N/mm^2

Diameter of cylindrical specimen = 103mm

Sample No.	L/d ratio	Failure Load (KN)	Compressive Strength N/mm^2
1	2.01	60.0	7.2
2	1.99	55.0	6.60
3	2.01	44.5	5.34
4	2.02	47.0	5.64
5	2.07	36.0	4.32
6	2.02	39.0	4.68

Average Compressive Strength = 5.63 N/mm^2

Table 15 COMPRESSIVE STRENGTH RESULTS

Water-aggregate ratio = 0.200

Age of cylindrical specimen at test = 28 days

Compaction pressure used = 7.2 N/mm^2

Diameter of cylindrical specimen = 103mm

Sample No.	L/d ratio	Failure Load (KN)	Compressive Strength N/mm^2
1	1.97	56	6.72
2	1.89	50	6.00
3	1.94	50	6.00
4	1.80	62	7.44
5	1.80	56	6.72
6	1.85	48	5.76
7	2.00	63	7.56
8	1.86	58	6.96
Average Compressive Strength =			6.65 N/mm^2

Table 16 COMPRESSIVE STRENGTH RESULTS

Water-aggregate ratio = 0.201

Age of cylindrical specimen at test = 28 days

Compaction pressure used = 7.2 N/mm^2

Diameter of Cylindrical specimen = 103mm

Sample No.	L/d ratio	Failure Load (KN)	Compressive Strength (N/mm^2)
1	1.97	55.5	6.66
2	2.00	53.0	6.36
3	1.94	41.5	4.98
4	2.00	51.0	6.12
5	1.93	54.5	6.54
6	1.95	55.0	6.60
Average Compressive Strength =			6.21 N/mm^2

One observe that the highest avarage compressive strength was obtained with water-aggregate ratio of 0.200 with a value of 6.65 N/mm^2. However, one also observe that the average compressive strength ranges approximately around 6.00 N/mm^2.

(b) Compaction Pressure test

Having obtained the best water-aggregate ratio for the soilcrete, one is left with the question of the best optimum compaction pressure that could be used in producing the cylindrical specimens to give at least a high compressive strength. Tests were then carried out using the following compaction pressures to mould the cylindrical specimens, 1, 3, 5, 7, 9, 11, 13, 15, 17, and 20 N/mm^2. Because of the many various compaction pressures involved, at least three sample specimens were produced and tested in each compaction pressure and the average of the

compressive strength obtained as the compressive strength for that compaction pressure. The result are as presented in Table 17 below.

Table 17 COMPACTION PRESSURE RESULTS

Water - aggregate ratio = 0.200

Age of cylindrical specimen at test = 28 days

Diameter of cylindrical specimen = 103mm

Compaction Pressure (N/mm^2)	L/d Ratio	Average Compressive Strength (N/mm^2)
1	1.92	0.92
3	1.87	4.55
5	1.87	4.30
7	1.96	4.90
9	1.96	5.83
11	1.93	5.70
13	1.98	5.59
15	1.96	5.96
17	2.01	5.48
20	1.89	4.74

As from compaction pressure of 11 N/mm^2 water or trace of water was observed coming out of the sample in the mould indicating that the pressure may be too high. As at 20 N/mm^2 compaction pressure, the sample materials (mixture) was coming out of the mould indicating too much pressure. The graphs of

both the water-aggregate ratio test and that of compaction pressure tests are as presented in figures 14 and 15 respectively. From figure 15 the best compaction pressure is 9.0 N/mm^2.

The sample specimen for both Elastic modulus test and creep tests were then produced using a compaction pressure of 9.0 N/mm^2, water-aggregate in ratio of 0.200 and curing condition as specified in section 3.2.5 of this thesis.

3.3.2 PREPARATION OF SPECIMEN FOR E-MODULUS TEST

As earlier stated the specimens are purely soilcrete materials which is mainly one part of ordinary Portland cement to 12 parts of laterite by volume with water-aggregate ratio of 0.20.

(a) Type and Size of Specimen

In the E-modulus test, six specimen were selected and cut into sizes by means of rotating carborundun blades such that the surfaces are flat, horizontal and smooth. After the cutting, the specimens have the following dimension.

Diameter = 103mm

Length = 200mm

L/d = 2.

Out of the six specimens, three were selected

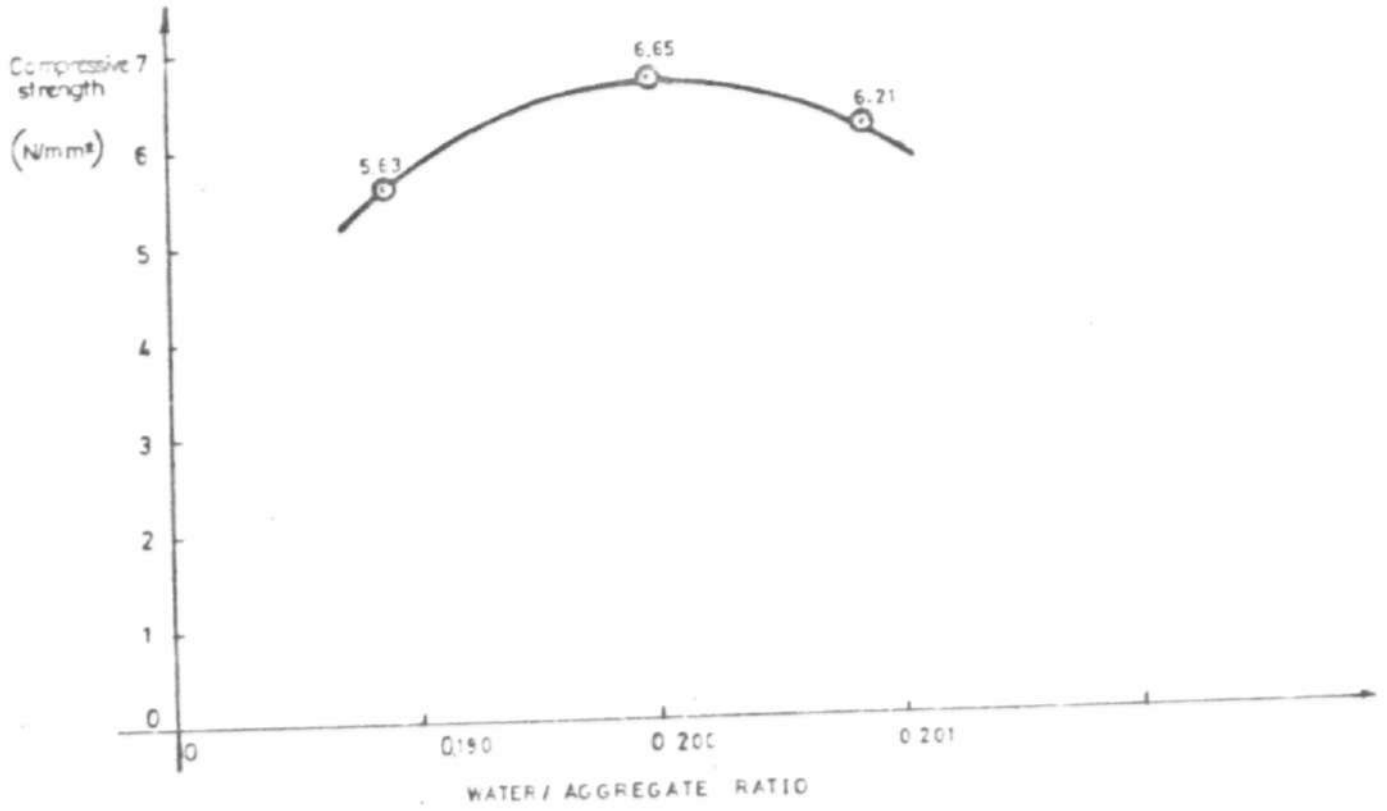


FIG 14 COMPRESSIVE STRENGTH VS WATER / AGG. RATIO

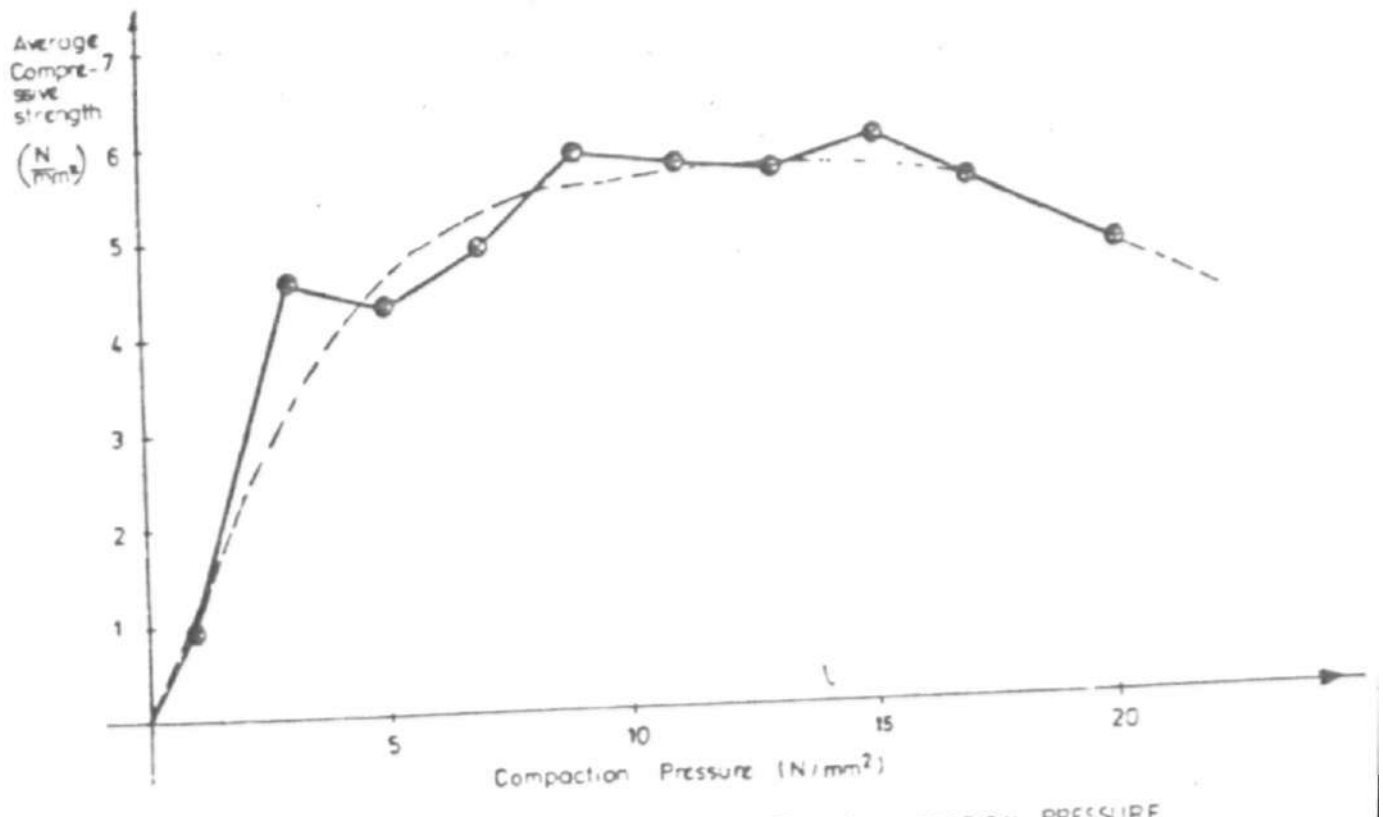


FIG 15 AVERAGE COMPRESSIVE STRENGTH VS COMPACTION PRESSURE

for compressive strength test to enable us know the ultimate strength of the soilcrete materials for the E-modulus test. All the specimens are of cylindrical shape. The results for the compressive strength test is as presented below.

Table 18 E-MODULUS COMPRESSIVE STRENTH TEST RESULTS

Sample No.	Diameter of Sample (mm)	Height of sample(mm)	Failure Load(KN)	Compressive strength N/mm^2
CS-01	103	200	102.5	12.30
CS-02	103	200	105.5	12.66
CS-03	103	200	110.0	13.20

Average Compressive Strength = 12.72 ≃ 13 N/mm^2

Lower load = = 0.02fsu = 0.254 N/mm^2 = 2.12KN

Upper Load = = 0.33fsu = 4.1976 N/mm^2 = 35KN

(b) Environmental Condition for Test

The environmental condition for the Elastic Modulus test is a room conditioned to a Relative Humidity of 65 per cent with a temperature of $20^{o}c$. The samples were put in the room for at least five hours before carrying out the test.

(c) Density and Moisture Content at Test

After conditioning the soilcrete samples to the environmental condition of 65 per cent Relative humidity and 20°c, the mosture content and the density of the material were immediately determined. The results are as presented below.

Table 19 MOISTURE CONTENT AT E-MODULES

Date of Test = 28 - 11 - 88

Place of Test = IWB - West Germany

Age of Specimen = 320 days.

Sample Number	EM-01	EM-02	EM-03
Wet weight + can (gm)	458.3	398.4	416.6
Dry weight + can (gm)	451.7	391.2	409.3
wet of can (gm)	121.8	111.1	102.9
Dry weight (gm)	329.9	280.1	306.4
Weight of Moisture (gm)	6.6	7.2	7.3
Moisture content (%)	2.00	2.57	2.38

Average Moisture Content at Test = 2.3%

Table 20 DENSITY AT E-MODULUS TEST

Sample Number	EM-01	EM-02	EM-03
Weight (kg)	3.090	3.110	3.100
Volume (m^3)	0.0016548	0.0016665	0.0016665
Density Kg/m^3	1867	1866	1860
Average Density = 1864 Kg/m^3			

Thus the moisture content and density of the sample specimens at test is 2% and 1864 Kg/m^3 respectively. The density of soilcrete material as determined in Nigeria is 1880 Kg/m^3. The difference of 16 could be due to the environmental condition at test.

3.3.3 APPARATUS FOR MEASUREMENT OF MODULUS OF DEFORMATION

The apparatus for the determination of elastic modulus consist mainly of displacement graph plotter, displacement detector, amplifier and walter + Balag digital electronic computerised measuring system. The block diagram of the connecting system for the test is as shown in figure 16.

On applying the force from the testing machine on the specimen, the displacement dectetor measures

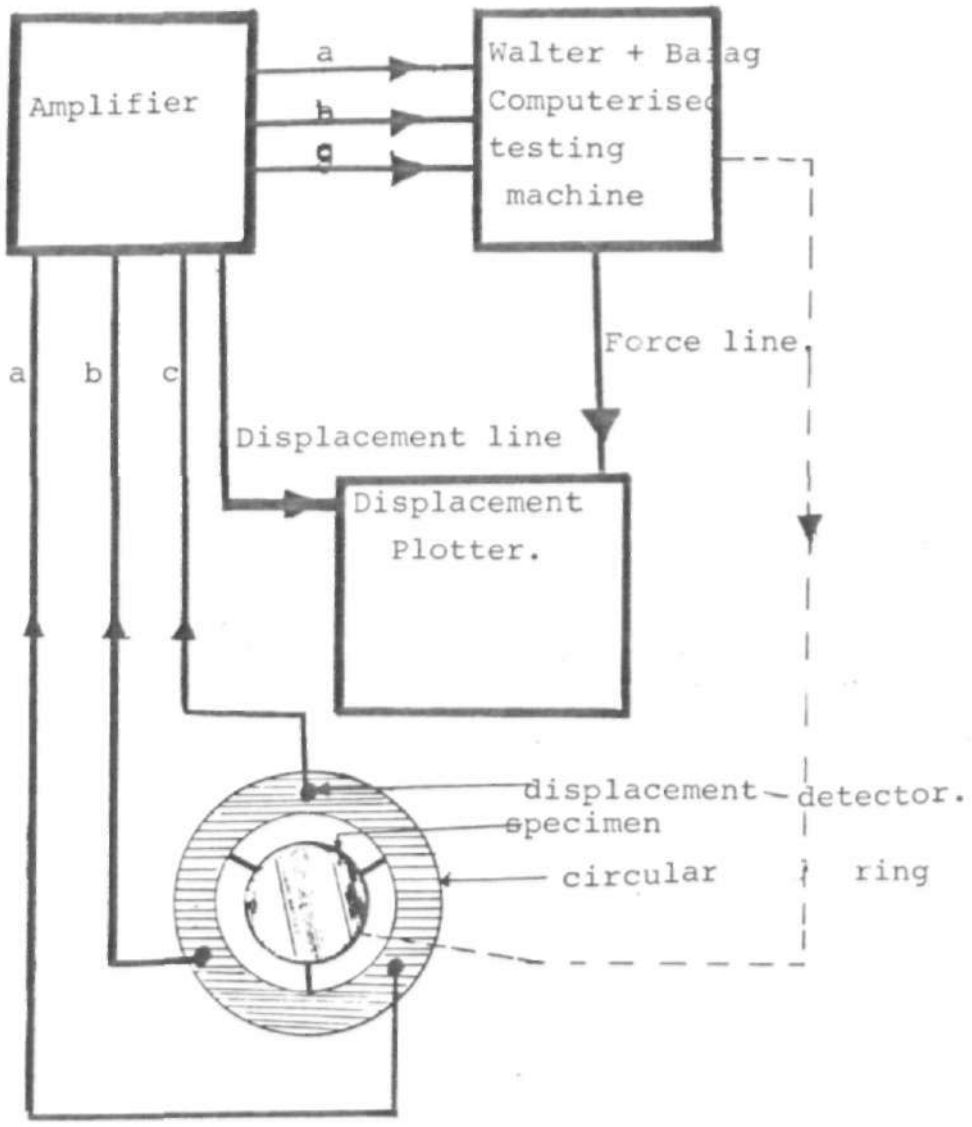


FIG. 16. BLOCK DIAGRAM FOR THE TESTING ASSEMBLY: (MODULUS TEST).

the strains which is amplified by the amplifier and gives the testing digital machine which then display the deformation in percentage of the guage length, in this case 100mm. The deformation from the amplifier and the applied force from the testing machine is communicated to the displacement plotter via the displacement and force lines respectively. This is displayed on the graph paper by the displacement plotter as the stress-strain graph. These graphs as plotted by the displacement plotter is shown in Section 3.3.4.

3.3.4 MODULUS OF DEFORMATION TESTS

From section 2.4.0, one observe that the German method of test for Modulus of Elasticity gives the best results. This is well described in the German Code DIN 1048. In this method of test, the specimen is loaded and unloaded ten times with a range of load of 0,02 σ_{max} to 0.33 σ_{max}. After repeated loading for about ten times, the strains measurements are taken at lower and upper loads. This process of repeated loading and unloading reduces the subsequent creep deformations that may be in addition to the elastic deformation. Thus the DIN 1048 method of determination gives an accurate value of E-modulus as strains are measured at small stress with loading and

unloading ten times.

The loadings are programmed into the digital compater measuring system in the following manner.

(a) Capacity of machine selected = 250KN

(b) Upper load for E-modulus = 35KN

(c) Lower load for E-modulus = 2.1KN

(d) Percentage for lower load programmed into the machine

$$= \frac{2.1}{250} \times 100 = 0.84\%$$

(e) Percentage for upper load programmed into the machine

$$= \frac{35}{250} \times 100 = 14.0\%$$

(f) Time required to load the specimen from lower load to upper load

$$= \frac{35.00}{2.08} = 16.7 = 17 \text{ seconds}$$

Thus loading and unloading takes 34 seconds.

(g) Time required to load the specimen to failure

$$= \frac{250}{2.08} = 120 \text{ second.}$$

The strains are measured at three points equally spaced along the periphery of the specimen and at middle length. The digital measuring machine displays the average value of the three points strains. The result as measured is as presented in Table 21 below.

Table 21 E-MODULUS TEST RESULTS

Date of Test = 28.11.88

Place of Test = IWB,West Germany

Age of test = 320 days

Sample Number	Stress and strain reading at first loading				Stress and strain reading after loading and unloading ten times				
	Lower Load		Upper Load		Lower Load		Upper load		E-modulus
	Stress N/mm^2	Strain $x10^{-3}$	Stress N/mm^2	Strain $x10^{-3}$	Stress N/mm^2	Strain $x10^{-3}$	Stress N/mm^2	Strain $x10^{-3}$	$\frac{\sigma u - \sigma L}{\varepsilon u - \varepsilon L}$ (N/mm^2)
EM-01	0.21	0.035	4.16	1.01	0.21	0.118	4.17	1.041	4290
EM-02	0.21	0.030	4.17	0.724	0.21	0.103	4.16	0.765	5967
EM-03	0.21	0.031	4.17	0.744	0.22	0.122	4.17	0.791	5904

The average E-modulus = 5387 N/mm^2

(h) Values of E-modulus as obtained from graph

Sample EM-01

$$\Delta\sigma = \sigma_2 - \sigma_1 = 4.176 - 0.216 = 3.96\ N/mm^2$$

$$\Delta\varepsilon = \varepsilon_2 - \varepsilon_1 = (1 - 0.1)\times10^{-3} = 0.9\times10^{-3}$$

$$E - Modulus = \frac{\Delta\sigma}{\Delta\varepsilon} = \frac{3.96}{9\times10^{-4}} = 4400\ N/mm^2$$

Sample EM-02

$$\Delta\sigma = \sigma_2 - \sigma_1 = 4.176 - 0.216 = 3.96\ N/mm^2$$

$$\Delta\varepsilon = \varepsilon_2 - \varepsilon_1 = (0.75 - 0.15)\times10^{-3} = 0.6\times10^{-3}$$

$$E\text{-Modulus} = \frac{3.96}{0.6\times10^{-3}} = 6600\ N/mm^2$$

Sample EM-03

$$\Delta\sigma = 3.96$$

$$\Delta\varepsilon = (0.8 - 0.15)\times10^{-3} = 0.65\times10^{-3}$$

$$E\text{-Modulus} = 6092\ N/mm^2$$

Average E-Modulus from graph = 5697 N/mm^2

It is worth noting that the value of the E-Modulus as obtained from graph may not be as accurate as that

given by the machine. The graphs as plotted by the displacement plotter is as shown in figure 17.

The ultimate strains as measure when the specimesns were loaded to failure are as shown in Table 22 .

Table 22 MEASURED ULTIMATE STRAIN

Sample Number	Ultimate strain $x10^{-3}$
EM - 01	8.05
EM - 02	5.90
EM - 03	7.50

Average ultimate strain = $\frac{(8.05 + 5.90 + 7.50)x10^{-3}}{3} = 7.14x10^{-3}$

The ultimate strain is about 2 times the ultimate strain of reinforced concrete. This implies that the soilcrete materials is about two times more plastic than concrete.

3.3.5 EXPERIMENTAL RESULT ANALYSIS AND EXPRESSIONAL RELATION

The average value of the secant Elastic Modulus as determined in this thesis is 5387 N/mm^2 while the average ultimate strain is 7.15 $x10^{-3}$ as compared with the values of reinforced concrete grade

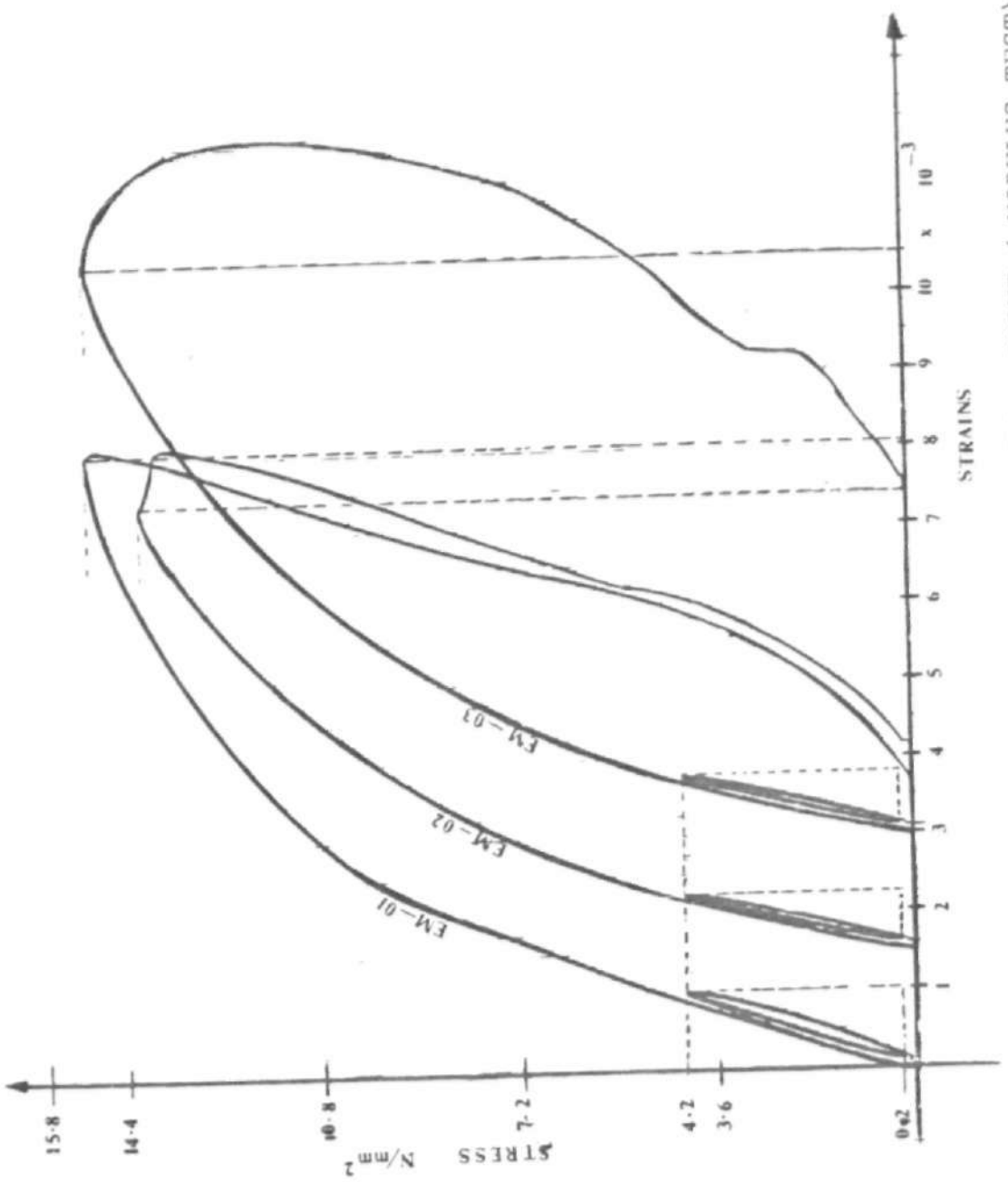


FIG. 17. SOILCRETE STRESS-STRAIN CURVE (MODULUS TEST).

20 of 25,000 N/mm^2 and 3.5 x10^{-3} respectively. In general terms, a wet specimen has a higher modulus of elasticity than dry one /281/ while dry high strength specimen has higher E-modulus than a lower dry specimen. The properties of aggregate also influence the modulus of elasticity, though generally they do not affect the compressive strength; the higher the modulus of elasticity of the aggregate, the higher the modulus of the resulting concrete. The shape of coarse aggregate particles /281/ and their surface characteristic may also influence the value of the modulus of elasticity of concrete and the curvature of the stress-strain relation.

It is worth noting also that the modulus of lightweight aggregate concrete is usually between 40 and 80 per cent of the modulus of ordinary concrete of the same strength. However, the modulus of light weight aggregates differs little from the modulus of the cement paste, thus mix proportions do not affect the modulus of elasticity of light-weight aggregate concrete. The relation between the modulus of elasticity and strength is unaffected by temperature of storage up to about 230^{o}c. However, for the same strength, the modulus is somewhat higher the lower the early curing temperature of the specimen. Thus steam-cured concrete has a lower modulus than moist-cured concrete of the same strength, though the difference is under ten per

cent. The modulus of deformation of concrete also depends on the moduli of deformation of its components like aggregate, the hardened cement paste and their proportions by volume in concrete. It increases with density and strength of the concrete /80/. The modulus of deformation of the available lightweight aggregates ranges mainly from 5,000 N/mm^2 to 1500 N/mm^2 /80/

Considering the fact that the E-modulus is a function of strength, age, aggregate, cement paste content, and density and the fact that the E-modulus of lightweight aggregate concrete is generally less than that of ordinary concrete; and soilcrete material is only a mix of 1 part cement to twelve part laterite by volume without actual aggregate as in lightweight or ordinary concrete, the value as obtained (5387 Nmm^2) is correct and accurate, bearing in mind the method of determination used. However, due to the limited number of test specimen, more further tests should be continued to really establish this value.

(b) Ultimate Strain

The magnitude of observed strains and the curvature of the stress-strain relation depends at times on the rate of application of stress. When loads are applied extremely rapid, for instance in less than 0.01 seconds, the amount of recorded strains

are greatly reduced and the stress-strain curve becomes extremely small. An increase in loading time from five seconds to about two minutes can increase the strain by up to 15 per cent. However, within the range of two to 10 minutes, the increase in strain as a result of loading time is very small and has no effect of the measured strains. The dependence of instantaneous strain on the speed of loading makes the demarcation between elastic and creep strains difficult. However, for practical purposes, an arbitrary distinction is made between the two strains; the deformation occuring during loading is considered elastic, while the subsequent increase in strain is regarded as creep. The modulus of elasticity satisfying this requirement is the secant modulus. Since repeated loading and unloading reduces the subsequent creep, the stress-strain curve on third or fourth loading is usally said to eliminate the creep strains. Thus loading and unloading ten times therefore completely removes the creep strains or at least practically. Measuring strains for a very small variations in stress also largely eliminate creep. In this thesis, a stress of $0.02\sigma_{max}$ (0.21 N/mm^2) was used as the lower stress for measuring strains.

Taking into account the fact that about six minutes were used for loading and unloading and

another extra two minutes for loading the specimen to failure; with lower stress of 0.21 N/mm^2 used in measuring strain, the average value of strain (7.15×10^{-3}) as measured is acceptable.

(c) Stress-Strain Graph for Soilcrete

On observing figure 17, it can be seen that the stress-strain relations for soilcrete, lightweight and ordinary concrete are similar, and the rectangular-parabola diagram of the CEB-FIP recommendation /80/ for ordinary concrete and lighweight concrete applies equally to soilcrete. The reason for the parabolic curve relation in the composite lies in the presence of interfaces between the cement paste and the lateritic materials and also in the development of microcraking at those interfaces. The cracks develop progressively at interfaces making varying angles with the applied stress; thus there is a progressive increase in local stress intensity and in the magnitude of strain. This result into strain increases at a faster rate than the applied stress, so that the stress-strain curve continues to bend over, with an apparent pseudo-plastic behaviour. However, the greater linearity of the ascending part of the stress-strain graph at low stresses is due to low or no internal microcracking at low stress before the high stress and failure.

This initial linear relationship is more pronounced in specimens of greater age in which the hardened cement paste is as strong as the aggregate used.

On observing carefully the part of the graph for loading and unloading, one can see that the path for loading and unloading is not the same, rather there is a gap or space between these two path (lines). In concrete even though the path are not exactly on the same line, there is no such wide gap between the paths. This may be due to the fact that the size of micro-pores in soilcrete materials is more than the size of micro-pores in the concrete. This could be caused by the fact that lateritic materials being more porous than sand and aggregates used in concrete. The gap is even more pronounced in sample EM-01 which also show more bending of the curve. The maximum loads and stresses at the failure of the samples are presented below.

Table 23 MAXIMUM STRESS AT FAILURE

Sample Number	Load at failure (KN)	Stress at failure (N/mm^2)
EM-01	124.2	14.91
EM-02	116.5	13.98
EM-03	124.0	14.88

(d) EXpressional Relations

There are various expressions for the stress-strain curve and expressions for the E-modulus as presented in section 2.4.0 of this thesis. Some of these expressions are tested with the experimental results obtained to see if the expression agree with soilcrete materials.

The following equations below has been checked and found not to satisfy the soilcrete materials.

Equation Number	Expression	Author	Reference
1	$Ec = 9.1\{fcu\}^{0.33}$	CP110:1972	/281/
2	$Ec = 1.7\ \rho^2\{fcu\}^{0.33}\ x10^{-6}$	CP110:1972	/281/
3	(Lightweight aggregate concrete) $Ec = 1.6\rho^2\{f_{cyl,k+8}\}\ x10^{-6}$	C E B	/281/
4	$Ec = 4.73\sqrt{fcyl}$	A C I	/281/
5	$Ec = 43\rho^{1.5}{}_{x}\ \sqrt{fcyl}\ x10^{-6}$	A C I	/281/

In general, all the equations above are of emperical form. Thus, one is not suprised that it does not agree with soilcrete materials.

However, some of the equations based on stress-strain curved as derived by various authors were tested and the various reults obtained is presented below.

(i) Desayi and Krishnan Expression /114, 115, 281/

The expression as developed by Desayi and Krishnan given as equation 6 is given below.

$$\sigma = \frac{E\varepsilon}{1 + \left[\frac{\varepsilon}{\varepsilon_o}\right]^2} \quad \text{- - - - - - - (6)}$$

where ε = Strain

σ = Stress

ε_o = Strain at maximum stress

E = Initial tangent modulus, assumed to be twice the secand modulus at maximum stress.

Thus $E = \frac{2\sigma max}{\varepsilon_o}$

The values of strains are obtained from the graph and the stress are calculated using the above expression and the values compared with that obtained from the graph. The check are as presented below.

(iii) Sample No. EM - 03; E = 3995

Strains $\times 10^{-3}$	Calculated stress (N/mm^2)	Stress as obtained from graph (N/mm^2)	Residuals N^2/mm^4
0	0	0	0
0.122	0.487	0.22	0.07
1.25	4.86	6.3	2.07
1.5	5.76	7.20	2.07
5.5	14.29	13.60	0.48
7.5	14.88	14.88	0

From the above tables, the sum of the residuals is 1.3 for samples EM-01 and EM-02 while that of sample EM-03 is 4.7. Generally, one would say that the equation of Desayi and Krishnan agrees with the soilcrete stress-strain curve, since the sum of squares of the deviation from the main curve is small. One observe also that at lower strains, for instance strains less than $1x10^{-3}$ or say $2x10^{-3}$ down, the deviation is high. This may be due to the fact that at lower stresses, there is little or no instantaneous deformation and also due to the sensitivity of the strain guage seating at lower stresses at the initial points of the graph. However, this improves as the stresses increases. The disadvantage of this expression however is that maximum strain

depends on the rate of loading.

(3) Carreira and Kuang-Han Chu Expression /79/

The curve equation as proposed by Serpentine was proposed by Carreira and Chu in a general form. Parameters which are functions of the material type and depends on the shape of the stress-strain curve is also proposed. The expression is as given below.

$$\frac{\sigma}{\sigma_{max}} = \frac{\beta\left(\frac{\varepsilon}{\varepsilon_{max}}\right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_{max}}\right)^{\beta}}$$ reffered to as equation 7

where $\beta = \frac{1}{1 - \left(\frac{\sigma_{max}}{\varepsilon_{max} E}\right)}$ = material parameter which depends on the shape of the graph.

σ = Stress

σ_{max} = Maximum stress at failure

ε = Strain

ε_{max} = Strain at Peak load (failure)

E = Initial tangent modulus

$$= \frac{2\sigma_{max}}{\varepsilon_{max}}$$

$$\beta = \frac{\sigma_{max}}{32.4} + 1.55$$ also

Maximum average strain as obtained from soilcrete material after the experiment = 0.00715

Average maximum stress = 14.6 N/mm^2

Average initial tanget modulus = 4146 N/mm^2

Therefore the value of B for soilcrete materials is

$$B = \frac{1}{1 - \frac{14.6}{0.00715 \times 4146}} = 1.9705$$

obtaining B using the second expression gives

$$B = \frac{14.6}{32.4} + 1.55 = 2.001$$

Taking B = 2 will give the general proposed curve as below

$$\frac{\sigma}{\sigma_{max}} = \frac{2\left(\frac{\varepsilon}{\varepsilon_{max}}\right)}{2-1+\left(\frac{\varepsilon}{\varepsilon_{max}}\right)^2}$$

$$= \frac{2\left(\frac{\varepsilon}{\varepsilon_{max}}\right)}{1+\left(\frac{\varepsilon}{\varepsilon_{max}}\right)^2}$$

Thus $\sigma = \frac{2\left(\frac{\sigma_{max}}{\varepsilon_{max}}\right)\varepsilon}{1+\left(\frac{\varepsilon}{\varepsilon_{max}}\right)^2} = \frac{E\ \varepsilon}{1+\left(\frac{\varepsilon}{\varepsilon_{max}}\right)^2}$

Therefore $\sigma = \frac{E\ \varepsilon}{1+\left(\frac{\varepsilon}{\varepsilon_{max}}\right)^2}$; this is the the same

expression as proposed by Desayi and Krishnan. The same table and observation for the expression also apply for the Carreira and Chu.

(4) Baldwin and North Expression /28/

Baldwin and North reviews some data on the effects of temperature up to 700°c upon the relationship between stress and strain for concrete under compression. They developed an expression inco-operating the effect of temperature. In general form, the expression is

$$\frac{\sigma}{\sigma_{max}} = f\left(\frac{\varepsilon}{\varepsilon_{max}}\right) \text{ where}$$

f is a functional form independent of temperature

σ = stress

σ_{max} = maximum stress

ε_{max} = maximum strain or strain at peak load.

However, the expression which the authors used is said to be communicated to them by P.H. Thomas personally and is given below.

At high temperature, the stress-strain curve is also distinctly non-linear and may be represented

by $\frac{\sigma}{\sigma_{max}} = \frac{\varepsilon}{\varepsilon_{max}} \exp\left[1 - \frac{\varepsilon}{\varepsilon_{max}}\right]$ ..... 8 (a)

over the whole range of $\frac{\varepsilon}{\varepsilon_{max}} > 0$..

According to the authors, for the early part of the curve, for small values of $\frac{\varepsilon}{\varepsilon_{max}}$, the curve is approximately linear and then

$$\frac{\sigma}{\sigma_{max}} = K \cdot \frac{\varepsilon}{\varepsilon_{max}} \quad \text{where } K = \text{constant}$$

Rearranging

$\sigma = K\left(\frac{\sigma_{max}}{\varepsilon_{max}}\right)\varepsilon$ which gives us the initial tangent modulus depending on the value of K, multipy by the strains. Writing E for modulus of elasticity then

$$\sigma = E\varepsilon .$$

and $E = K\left(\frac{\sigma_{max}}{\varepsilon_{max}}\right)$. ..... 8 (b)

The constant K in the equation can be determined from the values of E and $\frac{\sigma}{\varepsilon_{max}}$ at room temperature which is derived from the slope of the stress-strain curve for small values of strain. The value of K

as obtain by Baldwin and North is 2.18. E here implies the tensile modulus of elasticity but there is no substantial difference between the tensile modulus of elasticity and compressive modulus of elasticity /281/ For this reason the expression (a).. and (b) are checked with the experimental results obtained.

First with the expression

$$\frac{\sigma}{\sigma_{max}} = \frac{\varepsilon}{\varepsilon_{max}} e^{A}; \quad A = \text{Exp.} \left(1 - \frac{\varepsilon}{\varepsilon_{max}}\right). \quad - \quad 8(a)$$

Table 25 EXPERIMENTAL CHECK ON EQ 8

(i) Sample EM-01

Ratio of stress $\frac{\sigma}{\sigma_{max}}$	Ratio of strain $\frac{\varepsilon}{\varepsilon_{max}} \times e^{A}$	Residuals
0.014	0.012	0.000004
0.28	0.298	0.00032
0.72	0.698	0.00048
0.966	0.978	0.00014
1.0	1.0	0

(ii) Sample No. EM-02

Ratio of stress $\frac{\sigma}{\sigma_{max}}$	Ratio of strain $\frac{\epsilon}{\epsilon_{max}} \cdot e^{A}$	Residuals
0.015	0.017	0.000004
0.298	0.3097	0.00014
0.876	0.936	0.0036
0.927	0.967	0.0016
1.0	1.0	0

(iii) Sample No. EM - 03

Ration of stress $\frac{\sigma}{\sigma_{max}}$	Ratio of strain $\frac{\epsilon}{\epsilon_{max}} \cdot e^{A}$	Residuals
0.015	0.016	0.000001
0.423	0.384	0.0015
0.484	0.445	0.0015
0.914	0.957	0.0018
1.0	1.0	0

Observing table 25 (i), (ii) and (iii) one can see that the sum of the residuals; 0.000944, 0.0053 and 0,0048 are small. Thus the equation agree with the curve of soilcrete stress-strain curve. The compressive strength of concrete remains nearly constant until around 300 to 400^{o}c then there is

a dramatic decrease in strength. From his analysis the strength of the concrete at $700^{o}c$ is reduced to about 20 to 30 per cent and the strains at maximum stress at $700^{o}c$ is reduced to 40%. Since **E** is a function of maximum stress and maximum strain (equation 8(b)), it implies that the value of E for soilcrete should reduce to about 30 to 40 percent when the equation is checked with the experimental results. In other words, the value of E for soilcrete **at** $700^{o}c$ should lie within 30 to 40 percent. Below is the check for the equation.

$$E = K\left(\frac{\sigma max}{\varepsilon max}\right) \quad \ldots\ldots\ldots\ldots \quad 8(b)$$

Table **26** EXPERIMENTAL CHECK ON EQ 8(b)

Sample Number	Vale of E at $23^{o}c$	Value of E at K =1	value of E@K=2.18	Percentage of Ek=1(%)	Percentage Ek=2.18
EM-01	4290	1852	4038	43	94
EM-02	5967	2369	5165	39.7	87
EM-03	5094	1984	4325	38.95	85

On expressing the value of E-modulus at K=1 and K=2.18 as a percentage of that at room temperature, one observe that the value at K=1 has an average of the three sample of 40.44 per cent while that of K=2.18 is 89 per cent. From the work of Baldwin and North, that of K=1 is the correct value, thus soilcrete has

the value of K=1 instead of K=2.18. Therefore, for soilcrete material, the E-modulus at about 700°c is 40 per cent that of room temperature. Since there is no substantial difference between the tensile and compressive modulus, the soilcrete has at a temperature of 700°c a tensile modulus of 40 per cent of that room temperature which is about 2068 N/mm^2. Thus the expression becomes

$$E = \frac{\sigma_{max}}{\epsilon_{max}}.$$

This is also to say that the soilcrete material has an initial tangent modulus of 50 percent that of initial tangent modulus at room temperature.

Considering the equation below,

$$\frac{\sigma}{\sigma_{max}} = \frac{\epsilon}{\epsilon_{max}} \text{ Exp} \left(1 - \frac{\epsilon}{\epsilon_{max}}\right)$$

a graph of the ratio of stress is plotted against the ratio of strains for each sample and for all the samples. The graph are as shown in Figures 18 and 19 below.

One can observe that the rectangular parabola is maintained in all cases.

Fanella and Naaman Expression /133/

The expression of Fanella and Naaman characterize

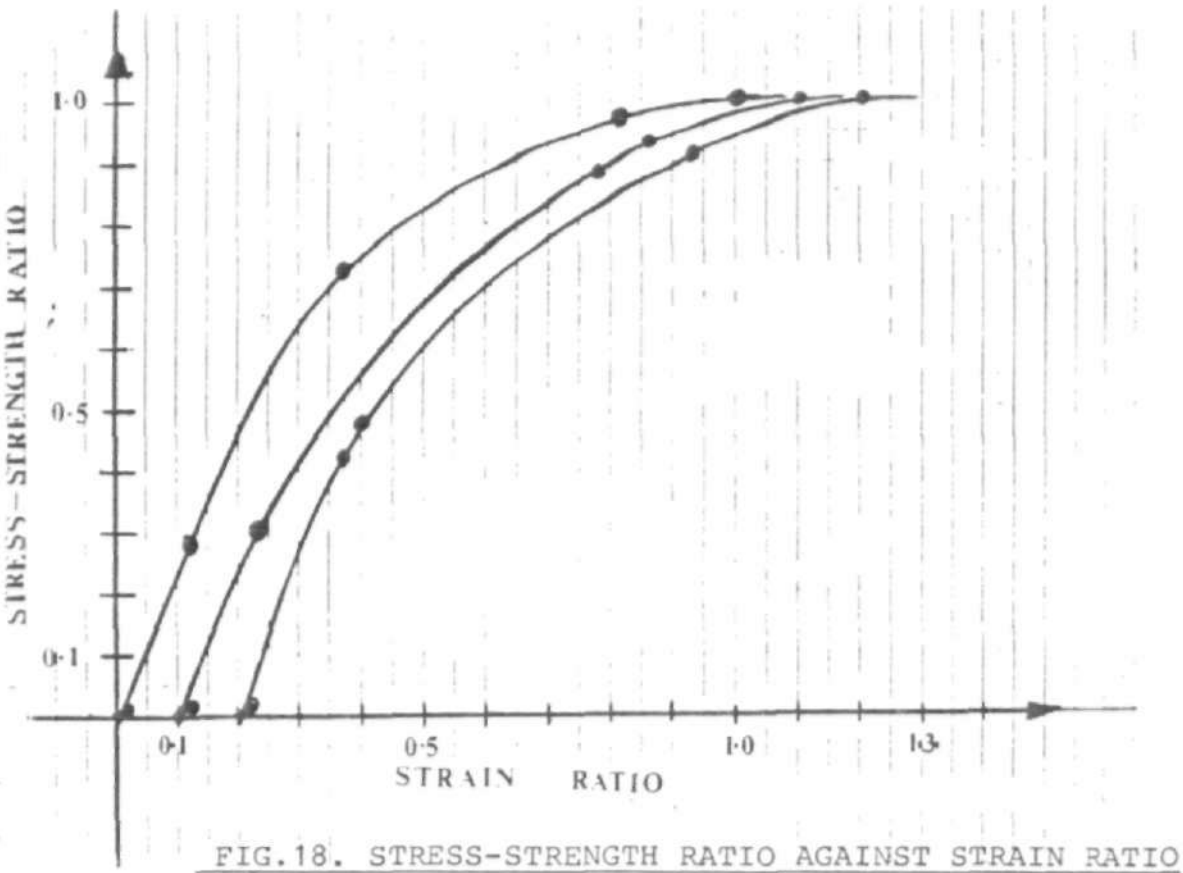


FIG.18. STRESS-STRENGTH RATIO AGAINST STRAIN RATIO

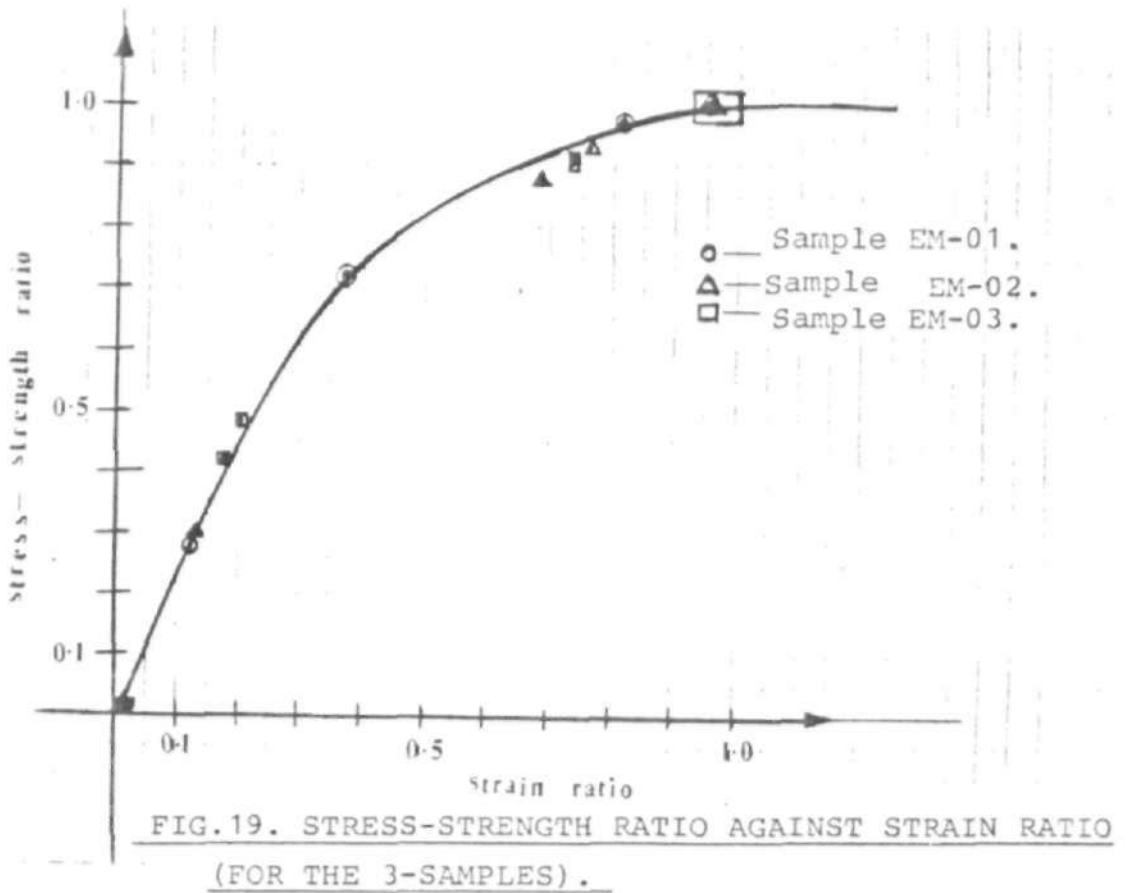


FIG.19. STRESS-STRENGTH RATIO AGAINST STRAIN RATIO (FOR THE 3-SAMPLES).

the stress-strain curve for fibre reinforced concrete mortar in compression. The fibre reinforced concrete is mainly a mixture of cement, sand and steel, glass monofilament polypropylene and twisted polypropylene fibres in various volume fraction. They used the analytical expression developed by sargin for uniaxial compression of concrete, which was later modified by Wang et al and Ahmad for biaxial and triaxial compression. After their analysis and testing with experimental results they arrived at equation no 9 given below.

$$Y = \frac{AX + BX^2}{1 + CX + DX^2} \quad \text{..... [9]}$$

where Y = stress ratio = $\frac{\sigma}{\sigma_{max}}$

X = strain ratio = $\frac{\varepsilon}{\varepsilon_{max}}$

A, B, C, and D are constants to be determined from the following boundary conditions of the curve

(1) $\frac{\partial Y}{\partial X}$ = A when X = 0 and Y = 0, $A = \frac{E_{(0.45)}}{E_{max}}$

Thus at origin slope = initial modulus.

(2) At Y = 0.45; X = 0.45; curve pass through $0.45\,\sigma_{max}$

(3) At Y = 1, X = 1; Curve passes through the peak load

(4) $\partial Y/\partial X$ = 0 at X = 1 and Y = 1 implies slope = 0 at peak load - (horizontal)

$A = \frac{E_{(0.45)}}{E_{max}}$ where $E_{(0.45)}$ = secant modulus at 0.45 σ_{max}; E_{max} = second modulus at peak load.

Using the four conditions, the four constant are obtained as below.

$$Y = \frac{AX + BX^2}{1 + CX + DX^2}$$

$$\frac{\partial Y}{\partial X} = \frac{(1 + C + DX^2)(A + 2BX) - (AX + BX^2)(C + 2DX)}{\{1 + CX + DX^2\}^2}$$

simplifying gives

$$\frac{\partial Y}{\partial X} = \frac{A + 2BX + BCX^2 - ADX^2}{(1 + CX + DX^2)^2}$$

Using the four condition and solving, the following solutions are arrived at

(1) $A = \frac{E_{(0.45)}}{E_{at\ peak}}$

(2) $B = D - 1$

(3) $C = A - 2$

(4) $D = -2.22A^2 + \left(0.45 - \frac{0.41}{A} + \frac{0.20}{A^2}\right) \times \frac{A^2}{0.11}$

$$D = -2.22A^2 + \left(0.45 - \frac{0.41}{A} + \frac{0.20}{A^2}\right) \frac{A^2}{0.11}$$

The constants are therefore determined using the experimental results.

(a) Sample No. EM-01

Secant modulus at 0.45 σ_{max} is

$$E = \frac{6.7095}{1.55 \times 10^{-3}} = 4329 \text{ N/mm}^2$$

Secant modulus at Peak load

$$E_{max} = \frac{14.91}{8.05 \times 10^{-3}} = 1852 \text{ N/mm}^2$$

$$\frac{E}{E_p} = \frac{4329}{1852} = 2.34$$

$A = 2.34$

$B = D-1$; $C = A-2 = 2.34 - 2 = 0.34$

$$D = -2.22(2.34)^2 + \left[0.45 - \frac{0.41}{2.34} + \frac{0.20}{(2.34)^2}\right] \times \frac{(2.34)^2}{0.11}$$

D = 3.34

B = D -1 = 3.34 - = 2.34

therefore

A = 2.34
B = 2.34
C = 0.34
D = 3.34

Thus $Y = \frac{2.34\ (X + X^2)}{3.34\ \{0.3 + 0.1X + X^2\}}$

(b) Sample No. EM-02

Secant modulus at $0.45\ \sigma_{max}$ = 5470 = E

E_p = 2369
A = 2.31
B = 2.22
C = 0.31
D = 3.22

$$Y = \frac{2.31\ (X + 0.96X^2)}{3.22\ \{0.31 + 0.1X + X^2\}}$$

the same as that of sample No. EM-01.

(c) Sample No. EM-02

E = 5151
E_p = 1984
A = 2.596
B = 3.78
C = 0.597
D = 4.78

$$Y = \frac{3.78\ (0.7X + X^2)}{4.78\left\{0.21 + 0.12X + X^2\right\}}$$

Thus on the average, the equation can be approximated as shown below for soilcrete.

$$Y = \frac{3\ (X + X^2)}{4\left\{0.3 + 0.1X + X^2\right\}}$$

Each of the equation as determined for each sample is checked with the experimental results to see how good it fitted. The ratio of the stress (Y) is taken and the corresponding ratio of strains with the constants are obtained (R.H.S) and compared as shown for each sample below.

Table 27 EXPERIMENTAL CHECK FOR EQ .9

(i) Sample No. EM-01

Values of Y	Values of the R.H.S. equations	Residuals
0	0	0
0.014	0.0283	0.000204
0.28	0.2888	0.000077
0.72	0.749	0.00084
0.97	0.9895	0.00038
1	1	0

(ii) Sample No. EM-02

Values of Y	Values of the R.H.S equation	Residuals
0	0	0
0.015	0.03967	0.000609
0.304	0.3086	0.0000217
0.876	0.96146	0.0073
0.927	0.98195	0.00302
1	1	0

(iii) Sample No. EM-03

Values of Y	Values of the R.H.S. equation	Residuals
0	0	0
0.015	0.0421	0.00073
0.423	0.4372	0.000207
0.484	0.5116	0.000762
0.914	0.9822	0.0047
1	1	0

One observe here also that at low stresses, the equation is not exact with the graph but within acceptable limit. This behaviour is as a result of the linearity of the graph at the early part which

is as a result of little or no micro-cracking and strain seating as earlier explained. Thus the soil-crete stress-strain curve agrees with the equation of stress-strain equation of Fibre Reinforceed mortar in compression but with different constants for A, B, C, and D for soilcrete.

(b) Desayi, Iyengar and Reddy Expression /114/

Desayi et al proposed an equation for the stress-strain curves of concrete confined in a circular steel spirals subjected to a monotonic compression. The proposed equation is as given in equation 10 and given below.

$$\sigma = \frac{A}{1 + B\varepsilon + C\varepsilon^2 + D\varepsilon^3} \quad \ldots\ldots\ldots\ldots \quad (10)$$

where σ = compressive stress

ε = compressive strain

and A, B, C, D are parameters to be obtained from boundary conditions. On observing the expression carefully, one noted that the conditions for determing the constants as proposed by Desayi et al is exactly the same as Fanella and Naaman. The above equation was earlier proposed by Saenz /281/ but modified into the above form by Desayi et al. However, the above Desayi et al expression is the same equation as proposed by Fanella and Naaman but in the stress ratio form

instead of the above form. While Fanella and Naaman obtained the second modulus at 0.45 σ_{max}, Desayi et al obtained their secant modulus at 0.85 σ_{max}. Whichever point used does not however matter as the equation is the same. Thus the above stress-strain curve equation for confined and unconfined concrete with circular spirals agrees with the stress-strain curve for soilcrete materials. The Desayi et al expression was developed in 1978 while that of Fanella and Naaman was 1985, a difference of seven years.

(7) Oleszkiewicz Expression /292/

Oleszkiewicz in 1973 developed an expression for the stress-strain curve for lightweight aggregate concrete under external load of short duration. His expression is also similar to the former ones but with different form. The expression as presented in equation No. 11 is given below.

$$\frac{\sigma}{\sigma_{max}} = \frac{\epsilon}{\epsilon_{max}}\left(2 - \frac{\epsilon}{\epsilon_{max}}\right) \quad - \quad [11]$$

where

σ = stress

σ_{max} = maximum stress - ultimate stress

ϵ = strain

ϵ_{max} = strain at peak load.

The equation is therefore checked with the experimental results to see the fittness. This is presented in the Table **28** below.

Table **28** EXPERIMENTAL CHECK ON OLESZKIEWICZ EPRESSION

(i) Sample No. EM-01

Ratio of stress- $\frac{\sigma}{\sigma_{max}}$	Value of R.H.S. $\frac{\epsilon}{\epsilon_{max}}\left[2-\frac{\epsilon}{\epsilon_{max}}\right]$	Residuals
0	0	0
0.014	0.024	0.0001
0.28	0.226	0.0029
0.72	0.603	0.014
0.97	0.964	0.000036
1	1	0

(ii) Sample No. EM-02

Ratio of stress $\frac{\sigma}{\sigma_{max}}$	Value of R.H.S. $\frac{\epsilon}{\epsilon_{max}}\left[2-\frac{\epsilon}{\epsilon_{max}}\right]$	Residuals
0	0	0
0.015	0.034	0.00036
0.304	0.243	0.00372
0.876	0.896	0.0004
0.927	0.944	0.000289
1	1	0

(iii) Sample No. EM-03

Ratio of Stress $\frac{\sigma}{\sigma_{max}}$	Value of R.H.S $\frac{\epsilon}{\epsilon_{max}}\left[2-\frac{\epsilon}{\epsilon_{max}}\right]$	Residuals
0	0	0
0.015	0.032	0.000289
0.423	0.306	0.014
0.484	0.36	0.015
0.914	0.929	0.000225
1.0	1	0

Again in this expression one observe the same persistent little deviation at low stresses and strain as a result of linearity, little or no internal cracking at low stresses and strai. seating. However, the sum of the residuals are small, thus the stress-strain curve expression as developed for light-weight concrete aggregate by Oleszkiewicz agree with soilcrete stress-strain curve.

He went further to develope a straight line expression using the strength of the lightweight aggregate concrete and its strains. The relation is given below (equation 12).

$$\epsilon_{max} = \epsilon_u + \frac{1}{E_u}\sigma_{max} \quad \text{- - - -} \quad 12$$

where

$\dot{\epsilon}_{max}$ = maximum strain at peak load

σ_{max} = maximum stress at peak load

ϵ_u = lowest ultimate strain of a given concrete when its crushing strength approaches zero

E_u = is the constant modulus of ultimate deformation increment for a given concrete.

The expression is therefore test with soilcrete sample to determine the lowest ultimate strain of the soilcrete and the constant modulus of ultimate deformation increament for this particular soilcrete. A graph is plotted using the values given in the Table below.

Table 29 VALUES OF ULTIMATE STRAIN AND STRESS

Sample No	Ultimate strain $x10^{-3}$	Ultimate stress (N/mm^2)
EM-01	8.05	14.91
EM-02	5.90	13.98
EM-03	7.50	14.88

The plot of the graph is as shown in Figure 20. Observing the graph, one can see that the soilcrete material has the lowest ultimate strain as its crushing strength approaches zero of 4 $x10^{-3}$. According to the report of Oleszkiewicz, the ultimate strain of concrete and lightweight aggregate concrete as their crushing strength approaches zero are 2 $x10^{-3}$ and 3 $x10^{-3}$ respectively. The lowest ultimate strain is a function of the aggregate and matric used. Thus the value of 4 $x10^{-3}$ obtained for soilcrete is reasonable since it contains no coarse aggregate except laterite. The constant modulus of ultimate deformation increament for soilcrete as obtained from thegraph is 4850 N/mm^2. The values reported by Oleszkiewicz ranges from 30,000 for lightweight aggregate to about 60,000 N/mm^2 for ordinary concrete. The value as obtained for soilcrete is reasonable since the lightweight aggregate concrete to ordinary concrete contains aggregate and are more rich mix compared to soilcrete which contains 1 part cement to 12 parts laterite, without any coarse aggregate. However, these values have been established on the basis of limited number of test results and their further experimental verification would be desired.

(8) Zhen-hai and Xiu-Qin: Eligehausen and Sawade Expression/127, 319, 408/

It is interesting to notethat these two group

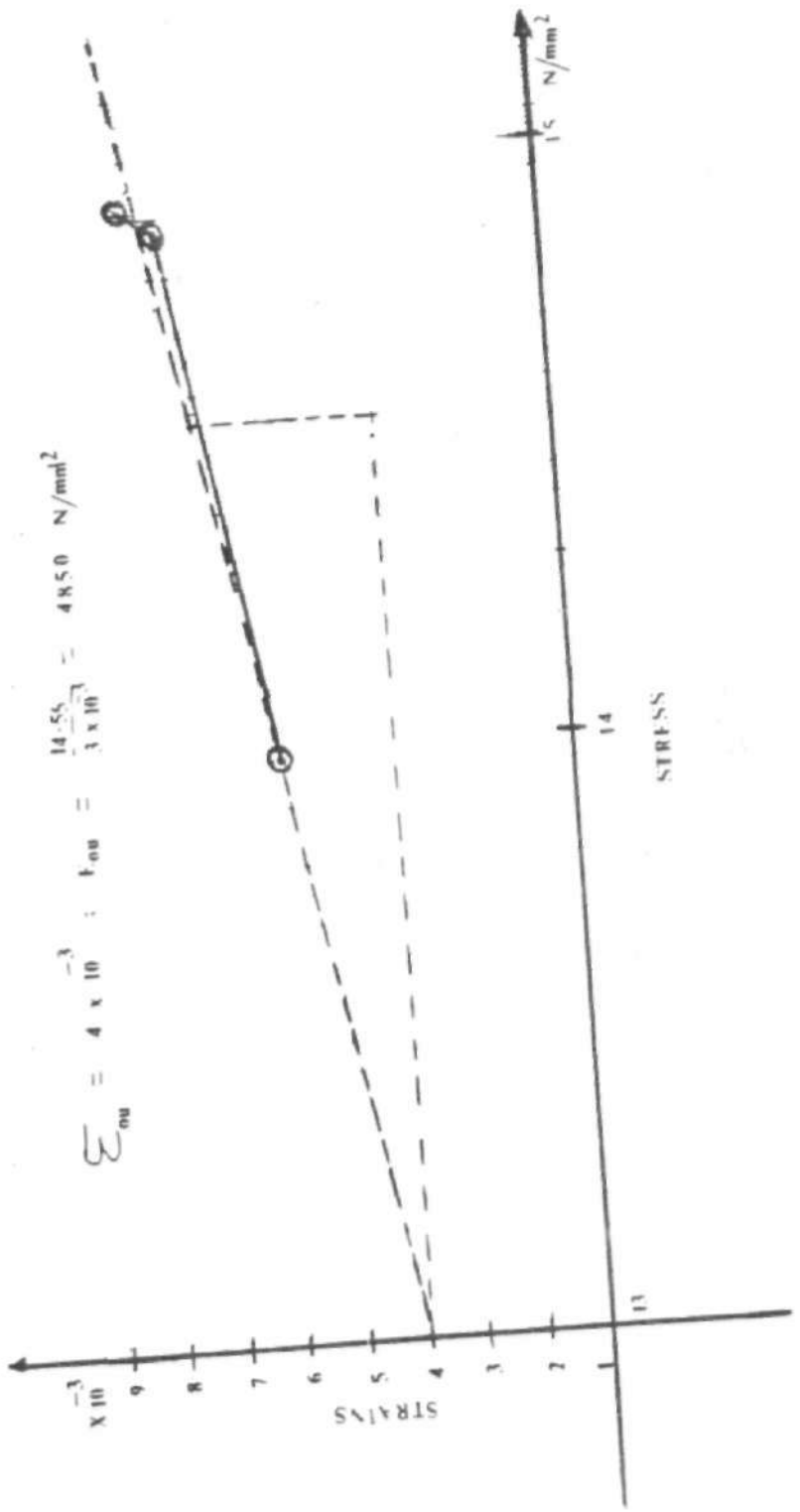


FIG. 20. ULTIMATE STRAIN AGAINST STRENGTH.

of researchers were working virtually on the same topic at the same time and arrive at the same results independently. While Professor Eligehausen and his group were working on the behaviour of concrete in tension at the Institut fuer Werkstoffe im Bauwesen, Stattgart, West Germany /127/; Professor Zhen-hai of Civil Engineering Department at the Tsinghua University Beijing, China and his group were working on investigation of complete stress-deformation curves for concrete in tension /407/. Even though the work of Zhen-hai and his group appeared in Journal of August 1987, their work dated back to earlier date. Professor Eligehausen and his group published their work in late 1985. Both groups working independently almost at the same time arived at the same conclusions, the same shape of stress-strain deformation curve of concrete in tension, the same failure mode of concrete in tension and the same shape of tensile strength versus crack width deformations.

However, while Eligehausen and his group generated an expression for the total energy needed for achieving complete seperation of the crack faces, the other group generated an expression for the complete stress-strain deformation for ascending and descending branch of the curve.

Since both the group arrived at the same graphs and conclusions, any of the equation generated can be patented to both groups.

In respect to this thesis, only the ascending expression for the stress-strain deformation of concrete in tension will be considered. As the ascending branch rises steeply at the beginning, a Polynomial with higher order is suggested as given in equation 12 (a) below.

$$y = A + BX + CX^2 + DX^6 - \qquad 12(a).$$

where

y = ratio of tensile strength to the ultimate tensile stress $= \sigma / f_t$

X = ratio of strain to strain at peak stress.

A, B, C, D, = constants to be determined from the following conditions below.

Four boundary conditions to be satisfied include:

(1) X = 0, y = 0

(2) X = 0, $\partial y/\partial x$ = Eo/Ep

where Eo = initial secant modulus of elasticity

Ep = Secant modulus at peak stress.

(3) X = 1, y = 1 - at peak load.

(4) X - 1, $\partial y/\partial x$ = 0 - at peak load.

Using the above conditions, with the fact that at this particular test for soilcrete materials, Eo/Ep = 2.; the constantsare obtained as follows

$$A = 0.$$

$$B = 2.$$

$$C = -1.$$

$$D = 0.$$

Thus for soilcrete materials, the equation becomes

$$y = 0 + 2X - X^2 + 0$$

$$y = 2X - X^2$$

simplifying further,

$$y = X(2 - X).$$

Bearing in mind that

$$y = \frac{\sigma}{\sigma_{max}}$$

$$X = \frac{\epsilon}{\epsilon_{max}}$$

The above equation developed by these group of researchers are exactly the same as that developed by Oleszkiewicz (equation Number 11), in the case of soilcrete materials

Thus, the check on Table 28 (i), (ii), (iii) holds for this equation and it agrees with soilcrete stress-strain envelope curve.

(a) DYNAMIC MODULUS

The emperical formular for concrete dynamic modulus as given by British Code CP110: 1972 expressed in GPa does not fit or agree with soil-crete material. Rather the formular given by the same Code but for lightweight aggregate concrete was tested and that seems to give a reasonable value. However, the accuracy of the value obtained below is not certain as modulus is a function of aggregate used in the mix also. The dynamic modulus as obtained from the expression is as shown below.

$$Es = 1.04\ Ed - 4.1 \quad \text{- CP110 1972}$$

Es for soilcrete $= 5387\ N/mm^2$

Sample EM-01

$$4290 = 1.04\ Ed - 4.1$$

$$Ed = \frac{4.290 + 4.1}{1.04} = 8070\ N/mm^2$$

Sample EM-02

$$Ed = 9680\ N/mm^2$$

Sample EM-03

$$Ed = 9620\ N/mm^2$$

$$\text{Average} = \frac{8070 + 9680 + 9620}{3} = 9122\ N/mm^2$$

Thus soilcrete has dynamic modulus of Elasticity of 9122 N/mm^2.

3.3.6 OBSERVATIONS

From available literature, there is a continuous flow of papers adding further formulae (models) for stress-strain curve description. This is in an attempt to inco-operate the small irregularities at the initial part of the curve. Popovic in his Survey in 1972 quoted about eighteen expressions including his own with various others that have given various expression since then. Yet with the various expression, the small irregularities at the initial part of the curve still persist. Thus the true regularities at the initial part of the stress-strain curve and the later part of the curve as a single mathematical expression is still a mystery. However, one can comfortably suggest that the stress-strain curve indicates a number of distinct stages in the state of the specimen (material), so that the stress-strain relationships cannot therefore be encompassed within a single formula. Thus the specimen (material) under short-term loading undergoes two stages: the elastic stage which gives the initial linear path of the curve and that of the non-linear path of the curve as a result of atrophying

(gradual failure effect) of microcracking. The elastic modulus of concrete or material under load is commonly regarded as a variable quantity. At the same time numerous studies have shown conclusively that its instantaneous value, whether measured under cyclic load or ultrasonically, remains practically constant and fairly close to its level for the early stages of loading. It is also known that under high-frequency load the non-linear process are not allowed enough time to affect the deformations so that the curve is linear under very short-term application of the load. Thus the problem of the elastic modulus can be therefore concluded by saying that the modulus is constant irrespective of the laad, and the non-linear behaviour is due mainly to atrophy. In reality therefore, the stress-strain curve describes not exactly the actual stress but rather the response (observed on the testing machine) of the specimens (materials) to the strains induced in them; the measured compressive force being divided by the norminal cross-section to yield the expected result. One can therefore suggest that the elastic modulus of specimens (material) coes not vary during the loading.

However, on practical applications and to some extent theoretical applications, the irregularities can be neglected and the various expressions can be used in practice to an acceptable degree of accuracy.

The deviations observed in most of the expressions are however very small and can be neglected.

From the tests carried out in this section and the analysis of the results, the following observations are made for soilcrete materials:

1) In soilcrete, the concept of water-cement ratio is not suitable, rather water-aggregate ratio is used. The best water-aggregate ratio for soilcrete production is 0.200.
2) The most suitable compaction pressure for soilcrete production is 9.0 N/mm^2.
3) The average cylinder compressive strength after 320 days of age is 14 N/mm^2.
4) The average E-modulus tested under 65 per cent relative humidity and 20°c for soilcrete materials is 5387 N/mm^2.
5) Average ultimate strain under the above environmental conditions for soilcrete materials is 7.15 $x10^{-3}$.
6) The average dynamic modulus is 9122 N/mm^2.
7) Most of the emperical relations as given by various investigators using density and compressive strength does not agree with soilcrete materials. More test is therefore required to establish such for soilcrete materials.

8) However, all the general expressions for the stress-strain curve envelope for concrete materials and lightweight concrete materials agree perfectly with the soilcrete materials.

9) The soilcrete materials have the lowest ultimate strain as its crushing strength approaches zero of $4.x10^{-3}$ ($4000x10^{-6}$).

10) The soilcrete materials have the constant modulus of ultimate deformation increament of 4850 N/mm^2.

3.4.0. LONG-TERM DEFORMATION TESTS

The term long-term deformation is mostly concerned with creep deformation. In other words, the change in strain deformation with time under constant load is often referred to as long-term deformation in this thesis. It is considered as a response to a constant sustained load and not any particular kind of inelasticity. It should be noted also that materials exhibit changes in strain with time when no external stress is acting. This is as a result of movement of water from or to the ambient medium and is known as shrinkage.

The long term deformation tests as carried out on the soilcrete materials, the apparatus used for the measurements and the analysis that follows are presented in this section of the thesis.

3.4.1. MATERIAL FOR EXPERIMENTS

The materials used for the experiments and its production are exactly the same as presented in section 3.3.1. of the thesis.

3.4.2. PREPARATION OF SPECIMENS

The specimens are purely soilcrete materials. Most research works as observed in

literature indicate that the ratio of the width or diameter of the testing specimen to its length is not important in creep tests, rather it is the cross-sectional size, shape and the type of loading that influence creep. Later reports even indicated that the cross-sectional size does not influence the actual basic creep and termed the desire for an overall characterization of creep and shrinkage in the cross-section as a source of large error as well as complexity as reported by the RILEM committee TC 69 of Septerber 1987. Thus various specimens of length diameter ratio of less than 2, equal to 2, between 2 and 4 and above 4 have been used. In terms of shape of specimen, concrete cyl..drical shape specimens exhibits the highest creep than the rest shape /43 /. However, there have been a lot of conflicting reports in area of size and shape.

(a) *SIZE OF SOILCRETE SPECIMEN*

The soilcrete specimens were cut into the following size for creep test.

Diameter = 103 mm

Length = 150 mm

For each environmental conditions two specimens were cut for creep test and two for corresponding

shrinkage measurements. The specimens were cut by means of a rotating carborundum blades equipment. This method of cutting ensures a plane horizontal surfaces that is completely flat. It also ensure uniform distribution of loads in the entire cross-section.

(b) PREPARATION OF SPECIMEN FOR STRAIN MEASUREMENT

As earlier noted, electrical resistance guages is not suitable for creep strains as a result of zero shift and creep of the bonding material between the guage and specimen. In this thesis therefore, mechanical guages are employed in all deformation measurements. All mechanical guages used in the long-term deformation measurement are of 4 millimetres and can measure up to one-thousandth of a millimetre.

In fixing the mechanical guages to the soilcrete specimens, the following are used;

1) a metal ring of inner diameter of 110 millimetres and external diameter of 150 millimetres and

2) stainless steel rod of diameter 8 millimetres and length 100 millimetres and bored solid metal nuts.

The metal ring, mechanical dial guage and the stainless steel rod are shown in A-56 and A-57 respectively.

Strain deformations are measured at three points equally spaced along the periphery of each specimen. The guage length is 100 millimetres along the length of the specimen and within the middle height. Each metal ring is fixed to the specimen by means of 5 millimetre diameter screw which is screwed on to the bored metal nuts of diameter 10 millimetres and thickness 3 millimetres and on to the specimen. Two metal rings are used for each specimen given the guage length of 100 millimetres. The stainless metal rods are fixed to the lower metal ring via the three point holes in the ring and screwed to position by a means of 4 millimetre diameter screw. The mechanical dial guages are fixed to the upper metal ring via the three point holes which are directly above the holes of the lower ones with stainless steel rods. The dial guages are fixed into position using also 4 millimetre diameter screw and making sure that it rest freely on the stainless rods and noting the zero reading. A prepared specimen for creep and shrinkage deformations is as shown in A-58
Before loading for creep, both prepared shrinkage and creep specimens are left in the environmental condition of test for 3-days; ususually from Friday

to Monday. This is to conditioned the specimen to the environmental condition for test. A minimum of five hours duration is normally required.

3.4.3. APPARATUS FOR MEASUREMENT OF CREEP DEFORMATION

An apparatus for creep tests should be able to maintain a constant known uniform stress distribution over the cross-section of the specimen with a minimum of maintenance and not requiring unduly heavy frame.

Three types of creep measuring apparatus were employed in this thesis. This include

a) Oil hydraulic measuring system of apparatus

b) Stabilized air hydraulic system and

c) Spring - loaded system of apparatus which has to be designed and constructed to suite the soilcrete specimen.

Since the dead-load method of loading system is hardly ever used, the three types of apparatus as mentioned above give rise to the three major loading methods employed in creep tests.

The discription of each of the apparatus and its application is given in the proceeding pages.

a) OIL HYDRAULIC MEASURING APPARATUS

This apparatus consist of (1) single stroke pump capable of pumping hydraulic oil at a rate of 4 litres per minute and a maximum pressure capacity of 315 bars, (2) an electronic equipment known as Regler (Herion Fluid Tronik) which is mainly a switch regulator and (3) about six loading creep frame of which only two was employed. Each loading frame consist of a lower loading ram, top floating plate, three plain mild steel tension rods of diameter 25 millimetres and rubber hoses connecting the various equipments making up the apparatus. The block diagram of the apparatus is as shown in figure 21.

As noted in most literature nowadays, the main problem of the hydraulic system of creep apparatus is the maintenance of a constant continuous sustained load because of sensitivity to small changes in the length of the specimen and also to the often unavoidable leakages of the hydraulic fluid. In middle December of 1988, creep tests were started using this creep apparatus. A constant load of 42 bars were applied to the creep specimen. It was observed that the apparatus cannot maintain a constant load, rather the load of 42 bars was oscillating between 41 and 54 bars.

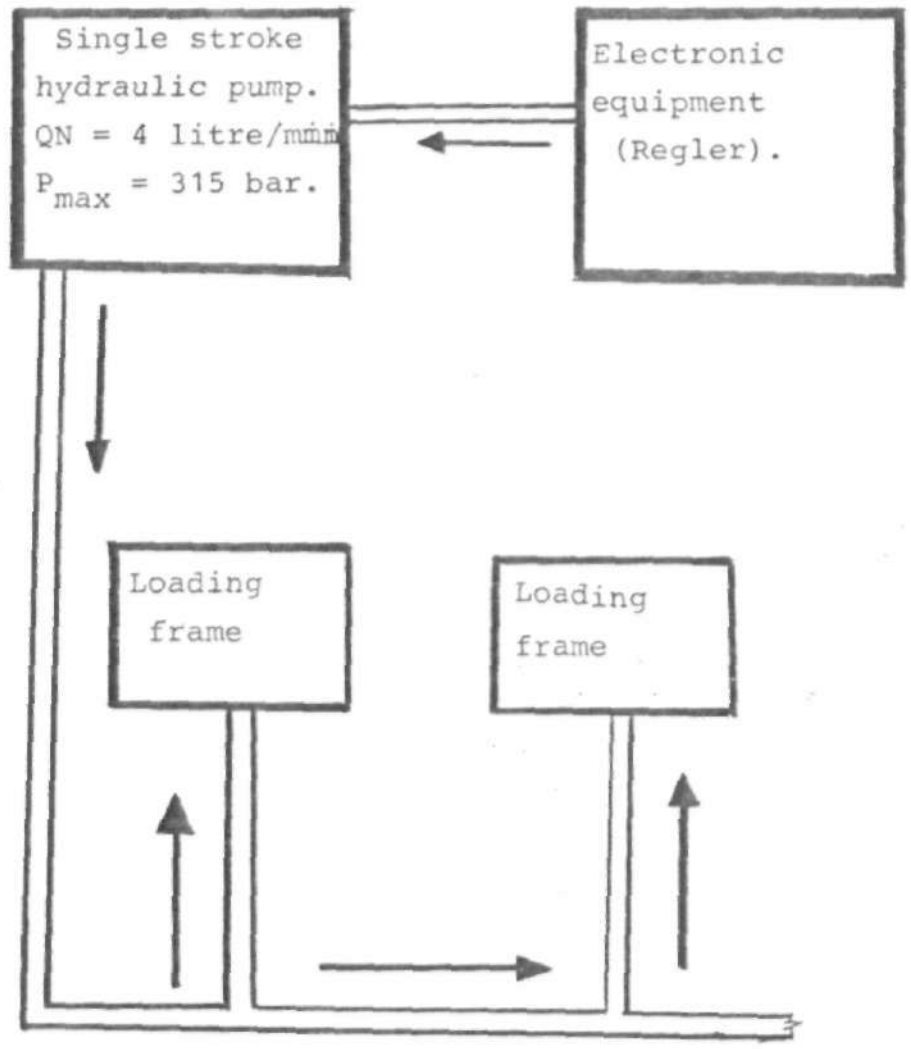


FIG. 21. BLOCK DIAGRAM FOR OIL HYDRAULIC SYTEM APPARATUS.

The load was applied on Monday and by Saturday night, one of the specimen was broken. The experiment was thus discontinued.

b) STABILIZED AIR HYDRAULIC SYSTEM

It consist of a compressor with a maximum capacity of 15 bar and can contain 10,000 litre of air, air valve, creep loading frames and steel pipes connecting the compressor to the valve and the creep frames. The block diagram is as shown in figure 22.

The creep loading frame consist of (1) a horizontal floating frame which can be moved vertically up and down, (2) screw nuts to fix the floating frame into position, (3) bottom circular loading ram connected to two rubber bags enclosed in a steel sheet, (4) maximum pressure indicator which returns the loading ram back when the maximum load has been applied and (5) Plastic rubber hoses connecting the steel pipe from compressor to the rubber bags and from rubber bags to maximum pressure indicator. The stability of pressure or load is brought about by the compressor unlike the single stroke pump. It also has two tension rods of diameter 23 millimetres and length 1000 millimetres. The various parts of the creep frame apparatus are as shown in

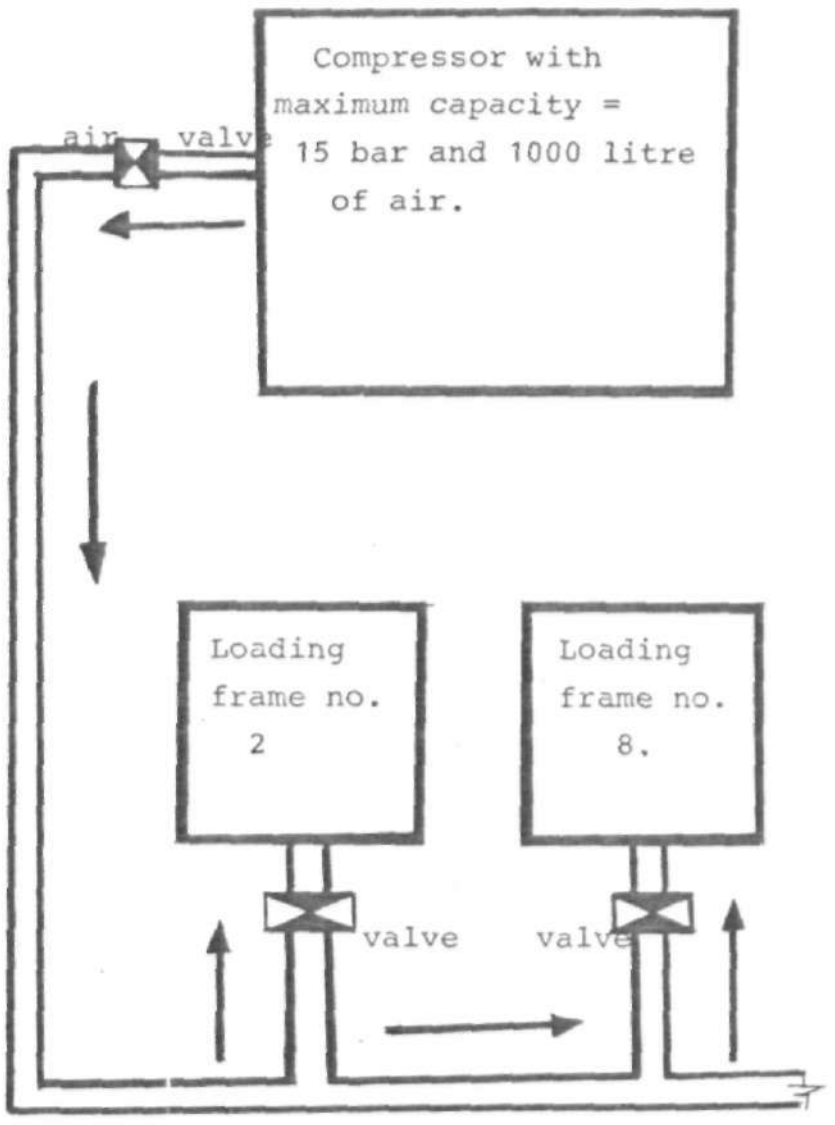


FIG. 22. BLOCK DIAGRAM FOR STABILIZED AIR HYDRAULIC SYTEEM

appendices A-59 and A-60 while the assembled creep frame apparatus is as shown in A-61 . The loading calibration graphs for creep frames number 2 and 8 which were used for soilcrete creep deformation test as carried out by IWB/FMPA, Construction division, university of Stuttgart West Germany are as shown in A-62 and A-63 respectively. It could be observed that a pressure of 6.2 bar and 6.1 bar were applied for creep frame number 2 and 8 respectively for a load of 35 kN. Maximum capacity of each creep frame is 10 bar. This creep apparatus was put to use with the soilcrete specimen in mid-December and its performance was very good. It maintained a constant stable load with time without much adjustment.

c) SPRING LOADED SYSTEM OF APPARATUS

Considering problems facing most developing countries, the use of compressor or single stroke pump to supply a constant load makes such experiment or research impossible to be carried out in such countries. As such, a simple but effective creep machine made from materials available in developing countries has to be developed, designed and constructed for the soilcrete materials. The designed and constructed soilcrete creep machine is purely made of steel materials which is readily abundant in Ajaokuta nd various other parts of Nigeria.

The creep machine consist of (1) a square bottom plate of length 200 millimetres and thickness of 30 millimetres, (2) a middle loading ram plate of the same size as bottom plate but of thickness 20 millimetres, (3) a top loading plate exactly the same size as bottom plate but with a hole of diameter 90 millimetres at the centre to accommodate the piston, (4) a hollow steel piston of inner diameter 35 millimetres and outer diameter of 90 millimetres to keep the springs in position, (5) circular steel plate springs with inner hole at the centre to slut into the piston, (6) nuts and washers and (7) four tension rods of length 500 millimetres and diameter of 11 millimetres. The various parts that make up the creep machine are as shown in A-64 , A-65 , A-66 and A-67 while the assembled creep machine is as shown in A-68

The use of the bottom plate is as a base for loading and to stabilize the entire frame. The loading ram plate, piston, circular plate springs and the top loading plate can be moved freely vertically up and down to accommodate the size of the specimen. Apart from keeping the circular springs in position, the cylindrical piston also keep the top loading plate in position. The top loading plate transfers the load via the springs to the middle loading ram plate which in turn distribute the load evenly to the

specimen. Circular steel plate springs are chosen because it can easily be cut into shape than forming a spiral spring especially in a developing country and its physical properties can easily be tested and known than a spiral spring. Secondly, because the stiffness of the spring is less than that of the soilcrete, the magnitude of the sustained load is only little reduced by the change in the length of the soilcrete specimen as a result of creep. Thus, the aim of the springs is to maintain a constant sustained load.

As observed in literature , one of the main problem with spring-loaded system is in the application of the proper load sufficiently rapid enough so that no creep takes place before the zero measurement is established. This problem is overcome by using a compression machine set into a capacity range of 100 kN for application of load via the top loading plate through the springs and the middle loading ram plate to the specimen. Thus, the process of application of large load being cumbersome is completely eliminated as the load is applied in less than a minute.

The creep machine was put into use in late December 1988 and its performance is very good. Apart from its being light and can easily be moved about, it maintained a constart stable load throughout.

CALIBRATION OF CIRCULAR PLATE SPRING

In any spring-loaded creep deformation apparatus, the springs are usually calibrated before putting into use. This is to enable one know the desired deformation with respect to the desired constant load under consideration at that time. In order to measure the deformation of the springs, mechanical dial guages are fixed via the top loading plate to rest on the middle loading ram plate. The arrangement is as shown in A-69 . Two 45 millimetre mechanical dial guages are fixed at the opposite ends along the diagonal of the top loading plate to rest freely on the middle loading ram plate. As the spring is loaded via the top loading plate, the top loading plate moves down with the dial guage and the deflection is indicated in the guage since the middle loading ram plate is supported by the lower platens of the compression machine during calibration and by specimen during creep test. Thus the two dial guages are permanently fixed to the machine to check the change in load during creep test.

Before deformation readings were taken for calibration of the springs, loads were run on the springs ten times to ensure uniform distribution. The maximum capacity of the springs

is 50 kN. However, at a load of 40 kN, the spring is about reaching its plastic stage. Two creep machines were produced on the whole and springs for each machine were calibrated by running the loads on the springs two times and the average displacements noted. A graph of load against average displacment is plotted for each machine and the displacement for the desired constant load for each machine noted. The results are as presented in tables 30 and 31 and figures 23 and 24 for each machine.

TABLE 30 . CALIBRATION READING FOR CREEP MACHINE NO.1

S/No.	Load (kN)	Displacement (mm) L/H dial guage	R/H dial guage	Average (mm)	Actual displacement (mm)
1.	0.5	0.89	0.93	0.91	-
2.	5	4. 30	4.30	4.30	3.39
3.	10	7.50	7.50	7.50	6.59
4.	15	11.05	11.06	11.06	10.15
5.	20	15.23	15.23	15.23	14.32
6.	25	19.88	19.89	19.89	18.98
7	30	25.26	25.26	25.26	24.35
8	35	31.32	31.34	31.33	30.42
9	40	38.61	38.65	38.63	37.72
Second loading of the same machine					
10	0.5	0.94	1.0	0.97	-
11	5	4.35	4.28	4.32	3.35
12	10	7.65	7.56	7.61	6.64
13	15	11.33	11.27	11.30	10.33
14	20	-	-	-	-
15	25	20.04	19.98	20.01	19.04
16	30	25.37	25.33	25.35	24.38
17	35	31.46	31.44	31.45	30.48
18	40	35.72	35.70	35.71	34.74

TABLE 31 . CALIBRATION READING FOR CREEP MACHINE NO.2

S/No.	Load kN	Displacements (mm) L/H dial guage	B/H dial guage	Average mm	Actual displacement (mm)
1.	0.5	1.13	1.05	1.09	-
2	5	4.21	4.02	4.11	3.02
3	10	7.51	7.28	7.39	6.30
4	15	11.24	10.97	11.11	10.02
5	20	15.17	14.88	15.03	13.94
6	25	19.77	19.47	19.63	18.54
7	30	24.91	24.61	24.76	22.67
8	35	31.0	30.70	30.85	29.76
9	40	35.12	34.83	34.98	33.89
		Second loading of the same machine			
10	0.5	1.21	1.14	1.17	-
11	5	4.27	4.07	4.17	3.0
12	10	7.60	7.34	7.47	6.30
13	15	11.13	10.90	11.02	9.85
14	20	15.17	14.88	15.03	13.86
15	25	19.71	19.42	19.56	18.39
16	30	24.95	24.65	24.80	23.63
17	35	30.94	30.62	30.78	29.61
18	40	35.15	34.85	35.0	33.83

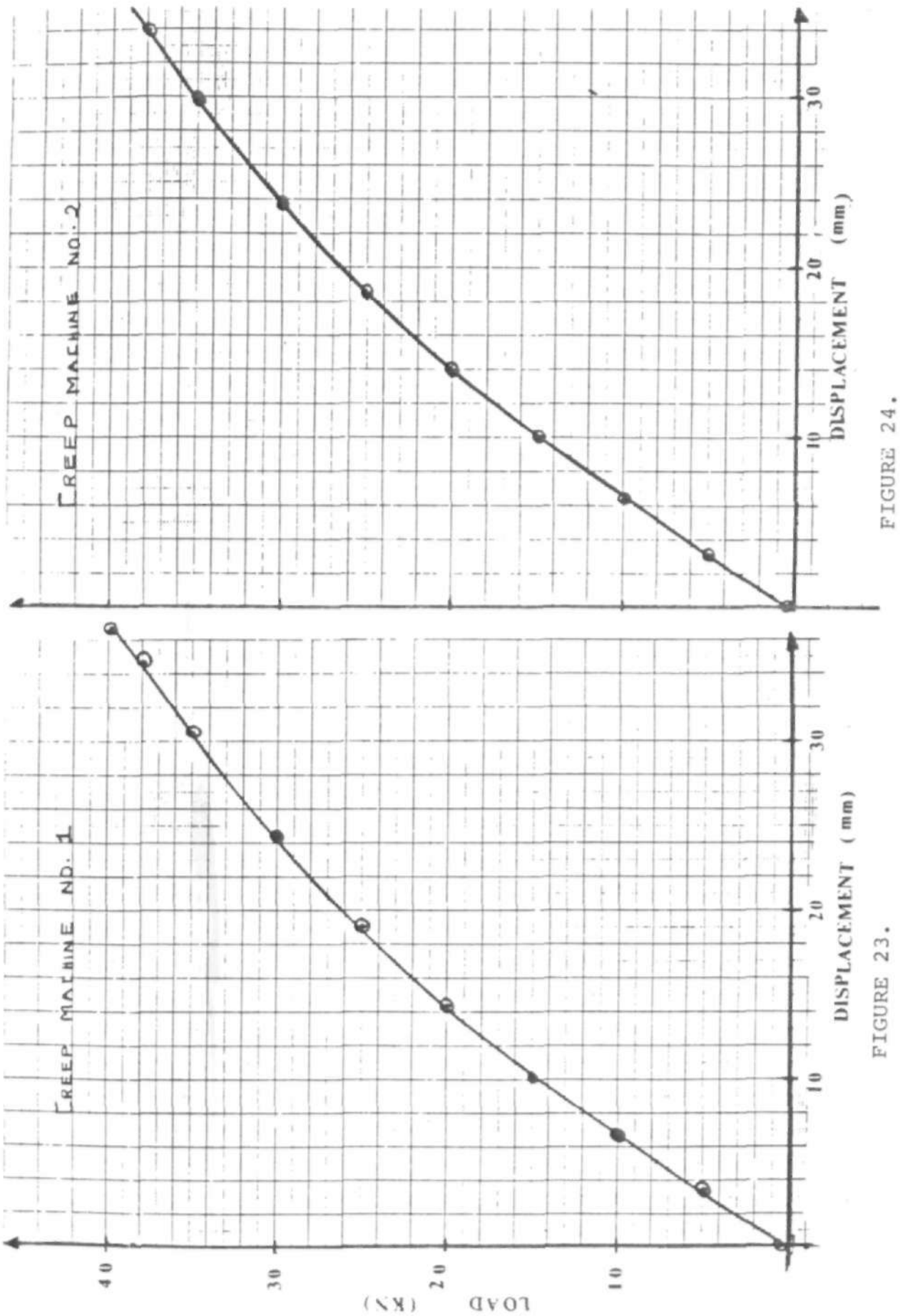


FIGURE 24.

FIGURE 23.

CALIBRATION GRAPHS FOR CREEP MACHINES

3.4.4. CREEP AND SHRINKAGE TESTS

In actual sense, real shrinkage tests were not performed, rather shrinkage deformations were measured alongside with the long-term deformations. Apart from the specimens being almost one year old before loading, the ambient environemtnal conditions were controlled, hence the shrinkage deformation measured is only used as a guide.

As observed in literature, extensive research work has been done on the value of the load to be applied for creep deformation tests. However, it is generally observed that creep is proportional to the stress-strength ratio for values up to 0.60 for concrete and lightweight concrete. Microcracking has also been observed in specimen at a stress-strength ratio values of between 0.40 to 0.60. This changes the actual creep value within the specimen. As such, Researchers found it more convenient to use the elastic load range which is about 33 per cent of the ultimate strength. The average ultimate load of soilcrete as determined in section 3.3.4 of this thesis is 105 kN (13 N/mm^2). Hence, the applied constant stable load used for creep is 35 kN (4.26 N/mm^2) for all tests.

(a) Environmental Conditions for Creep Deformation Tests

For soilcrete materials, the "standard" curing condition and the "Mass" curing condition as proposed by ASTM method of creep tests cannot be directly applied. This is because the best curing method for soilcrete materials as discussed in section 3.2.5 of this thesis is not in accordance with the moist curing in "standard curing condition" and also in "Mass curing condition". As a result of the lateritic materials in soilcrete, it requires a lot of water for hydration, as such the curing method as described in both "standard" and "Mass" curing conditions will certainly lead to cracks in the soilcrete specimens. Thus, the method of "other curing condition" as proposed by the same ASTM method of creep test was employed for the specimen. This "other curing condition is the same as described in section 3.2.5.

Two environmental conditions were employed for the long term defromation tests. This include

1) 50 ± 2 per cent relative humidity with a temperature of $23^{o}C$. This ambient condition is approximately a transition period from hot season to dry season in Nigeria which is usually either in October, November or December.

ii) 70 ± 4 per cent relative humidity and a temperature of 14°C. This is approximately cold season in Nigeria and is usually either in November, December and January or December, January and February. These environmental conditions are controlled and kept constant by means of a humidity machines. Two specimens each for long-term deformation and shrinkage deformation were tested under each environmental conditions.

b) Strain Measurements

The long-term deformations (strains) are measured in the following manner.

i) strain readings are taken immediately before and after loads
ii) every hour for the next six hours
iii) every day for one week
iv) every week for one month and
v) every month for ten months.

The measured deformations for both creep and shrinkage are as presented below for each environmental conditions. The raw data and the calculation of average long-term deformations are presented in Appendix A-70 to A-71 and A-72 to A-73 for the two environmental conditions respectively.

That of shrinkage is shown in A-74 and A-75 to A-77 for 50 per cent and 70 per cent relative humidity respectively. However, the actual total average deformation, the corresponding creep and the environmental condition at test are shown in table 32, 33, 34 and 35. The average corresponding shrinkage as measured under each environmental conditions is also indicated in each of the table above.

The average total deformation for each specimen is obtained from the three points long-term deformations measurements equally spaced along the periphery of each specimen. The creep deformation is obtained by substracting the deformation immediately after loading for each specimen. For each point measurement, the deformation itself is obtained by substracting the reading before loading otherwise known as initial reading and indicated as reading before loading. The shrinkage deformation is obtained in the same way as the normal deformation by taking note of the initial reading and subtracting it from the subsequent readings. One can observe that there is virtually no shrinkage at 50±2 per cent humidity where as in 70±4 per cent it was oscillating up and down as a result of the high humidity.

TABLE 32 TOTAL DEFORMATION, CREEP AND CORRESPONDING SHRINKAGE

Relative humidity = 50±2 per cent

Temperature = 23±1°C

Sample Number = 050 CP - 0 3

Creep machine No = 01

Serial Number	Time (days)	Total deformation x 10^{-5}	Creep deformation x 10^{-5}	Corresponding shrinkage x 10^{-5}
1	Deformation	-	-	-
2	after loading	73	-	-
3	0.042	76.1	3.1	0
4	0.125	76.3	3.3	0
5	0.167	76.4	3.4	0
6	0.208	76.4	3.4	0
7	0.250	77	4.0	0
8	0.750	78	5.0	0
9	1	78.7	5.7	0
10	2	79.8	6.8	0
11	3	80.7	7.7	0
12	4	81.6	8.6	0
13	5	81.8	8.8	0
14	6	87.3	14.3	0
15	7	87.9	14.9	0
16	14	91.7	18.7	0
17	21	94	21.0	0.35
18	28	96.8	23.8	0
19	56	109	36.0	0
20	84	115	42.0	0
21	112	120	47.0	0
22	140	128	55.0	-0.65
23	168	133	60.0	-0.65

TABLE 33 . TOTAL DEFORMATION, CREEP AND CORRESPONDING SHRINKAGE

Relative humidity = 50±2 per cent

Temperature = 23±1°C

Sample Number = 050 CP - 04

Creep machine No = 02

Serial Number	Time (days)	Total deformation $\times 10^{-5}$	Creep deformation $\times 10^{-5}$	Corresponding shrinage $\times 10^{-5}$
1	Deformation	-	-	-
2	after loading	69	-	-
3	0.042	71.8	2.8	0
4	0.125	72.3	3.3	0
5	0.167	72.7	3.7	0
6	0.208	72.7	3.7	0
7	0.250	73.3	4.3	0
8	0.750	75.3	6.3	0
9	1	76	7	0
10	2	77.7	8.7	0
11	3	78.4	9.4	0
12	4	79.7	10.7	0
13	5	80.8	11.8	0
14	6	81.5	12.5	0
15	7	81.9	12.9	0
16	14	86.8	17.8	0
17	21	91	22	0.35
18	28	93.3	24.3	0
19	56	103.3	34.3	0
20	84	111.7	42.7	0
21	112	118.7	49.7	0
22	140	124.6	55.6	-0.65
23	168	129.7	60.7	-0.65

TABLE 35 . TOTAL DEFORMATION, CREEP AND CORRESPONDING SHRINKAGE

Relative humidity = 70± 4 per cent

Temperature = 14°C

Sample Number = 70 CP - 02

Creep stand No = 2

Serial Number	Time (days)	Total deformation 10^{-5}	Creep Deformation x 10^{-5}	Corresponding shrinkage x 10^{-5}
1	deformation	-	-	-
2	after loading	75	-	-
3	0.042	79	4	0
4	0.125	81.7	6.7	1.7
5	0.167	82.3	7.3	2.9
6	0.208	82.3	7.3	2.0
7	0.250	82.7	7.7	2.9
8	0.750	84.8	9.8	2.9
9	1	85.2	10.2	2.0
10	2	88	13	2.0
11	3	90.3	15.3	2.9
12	4	93	18	2.9
13	5	95.7	20.7	2.9
14	6	96.7	21.7	2.9
15	7	97.2	22.2	5.4
16	14	98	23	2.2
17	21	103.3	28.3	2.5
18	28	105.8	30.8	1.7
19	56	122.3	47.3	6.4
20	84	126.7	51.7	3.0
21	112	131.5	56.5	-0.95
22	140	142.3	67.3	0.2
23	168	148	73	-3.3

3.4.5 EXPERIMENTAL RESULT ANALYSIS UND EXPRESSIONAL RELATIONS

The need to analyse any experimental data in any research work cannot be overemphasized. As such, this section of the thesis is to analyse the experimental data presented above and to check the various expresional relations given in chapter two section 2.4.5 for long-term deformations.

The average total deformation and the corresponding creep for the 50 per cent and 70 per cent relative humidity storage are shown in tables 36 and 37 respectively while the ratio of the creep deformation at 70 per cent to 50 per cent relative humidity is shown in table 38 . One observe that, initially about one hour after loading the creep deformation were the same but after about two hours under load, the creep deformation at 70 per cent relative humidity is two times that of 50 per cent R·H· of storage. The ratio is fairly kept constant for about a week under load after which the ratio sharply drops and tends towards the value of one with time under load.

The plot of the deformations against time and the specific deformations against time are shown in figures 25 to 30 of the respective samples and conditions of storage of the samples. The specific deformations are the total or creep deformations divided by the constant

TABLE 36 AVERAGE DEFORMATION AND CREEP UNDER 50 ± 2 PER CENT HUMIDITY

Serial Number	Time (days)	Deformations $X10^{-5}$		Average deformation $X10^{-5}$	Creep deformation $X10^{-5}$
		50CP-03	50CP-04		
1	Deformation after Loading	-	-	-	
2		73	69	71	-
3	0.042	76.1	71.8	74	3
4	0.125	76.3	72.3	74.3	3.3
5	0.167	76.4	72.7	74.6	3.6
6	0.208	76.4	72.7	74.6	3.6
7	0.250	77	73.3	75.2	4.2
8	0.750	78	75.3	76.7	5.7
9	1	78.7	76	77.4	6.4
10	2	79.8	77.7	78.8	7.8
11	3	80.7	78.4	79.6	8.6
12	4	81.6	79.7	80.7	9.7
13	5	81.8	80.8	81.3	10.3
14	6	87.3	81.5	84.4	13.4
15	7	87.9	81.9	84.9	13.9
16	14	91.7	86.8	89.3	18.3
17	21	94	91	92.5	21.5
18	28	96.8	93.3	95.1	24.1
19	56	109	103.3	106.2	35.2
20	84	115	111.7	113.4	42.4
21	112	120	118.7	119.4	48.4
22	140	128	124.6	126.3	55.3
23	168	133	129.7	131.4	60.4

TABLE 37 AVERAGE DEFORMATION AND CREEP UNDER 70 ± 4 PER CENT HUMIDITY

Serial Number	Time (days)	Deformation $\times 10^{-5}$			Creep deformation $\times 10^{-5}$
		70 CP-01	70 CP-02	Average	
1	Deformation after	=	=	-	-
2	Loading	102.7	75	88.9	=
3	0.042	105	79	92	3.1
4	0.125	110.5	81.7	96.1	7.2
5	0.167	111	82.3	96.7	7.8
6	0.208	111	82.3	96.7	7.8
7	0.250	111.5	82.7	97.1	8.2
8	0.750	115.3	84.8	100.1	11.2
9	1	115.8	85.2	100.5	11.6
10	2	119.3	88	103.7	14.8
11	3	120.7	90.3	105.5	16.6
12	4	123.3	93	108.2	19.3
13	5	126.8	95.7	111.3	22.4
14	6	128.5	96.7	112.6	23.7
15	7	128.8	97.2	113.0	24.1
16	14	129.8	98	113.9	25
17	21	134.8	103.3	119.1	30.2
18	28	138.5	105.8	122.2	33.3
19	56	156	122.3	139.2	50.3
20	84	164.7	126.7	145.7	56.8
21	112	168	131.5	149.8	60.9
22	140	182.3	142.3	162.3	73.4
23	168	188.7	148	168.4	79.5

TABLE 38 - RATIO OF CREEP AT 70 PER CENT HUMIDITY TO THAT AT 50 PER CENT

Serial Number	Time (Days)	70% R.H Creep $X10^{-5}$	50% R.H Creep $X10^{-5}$	Ratio 70/50
1	-	-	-	-
2	-	-	-	-
3	0.042	3.1	3	1.03
4	0.125	7.2	3.3	2.18
5	0.167	7.8	3.6	2.17
6	0.208	7.8	3.6	2.17
7	0.250	8.2	4.2	1.95
8	0.750	11.2	5.7	1.96
9	1	11.6	6.4	1.81
10	2	14.8	7.8	1.90
11	3	16.6	8.6	1.93
12	4	19.3	9.7	1.99
13	5	22.4	10.3	2.17
14	6	23.7	13.4	1.77
15	7	24.1	13.9	1.73
16	14	25	18.3	1.37
17	21	30.2	21.5	1.40
18	28	33.3	24.1	1.38
19	56	50.3	35.2	1.43
20	84	56.8	42.4	1.32
21	112	60.9	48.4	1.26
22	140	73.4	55.3	1.33
23	168	79.5	60.4	1.32

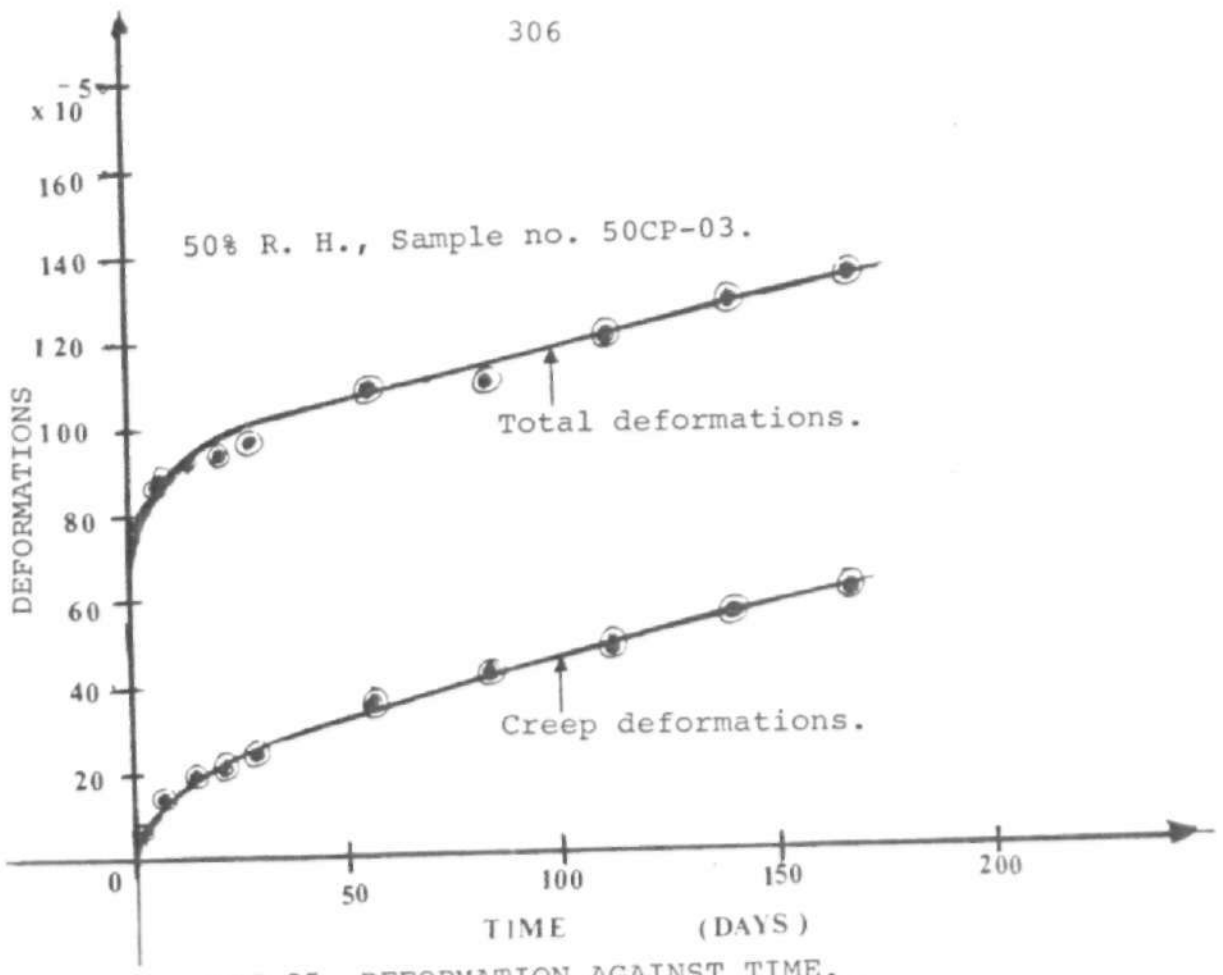


FIG.25. DEFORMATION AGAINST TIME.

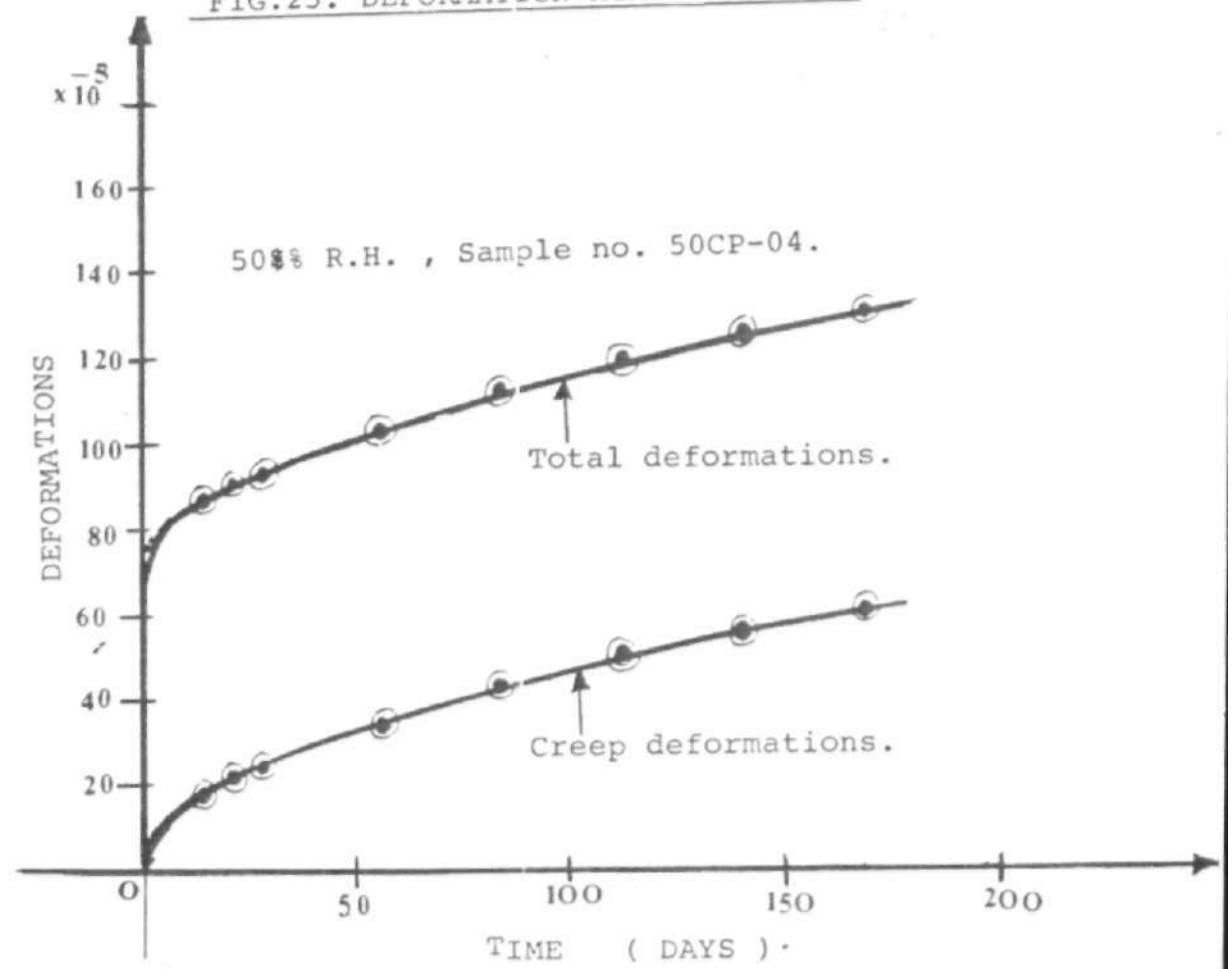


FIG.26. DEFORMATION AGAINST TIME.

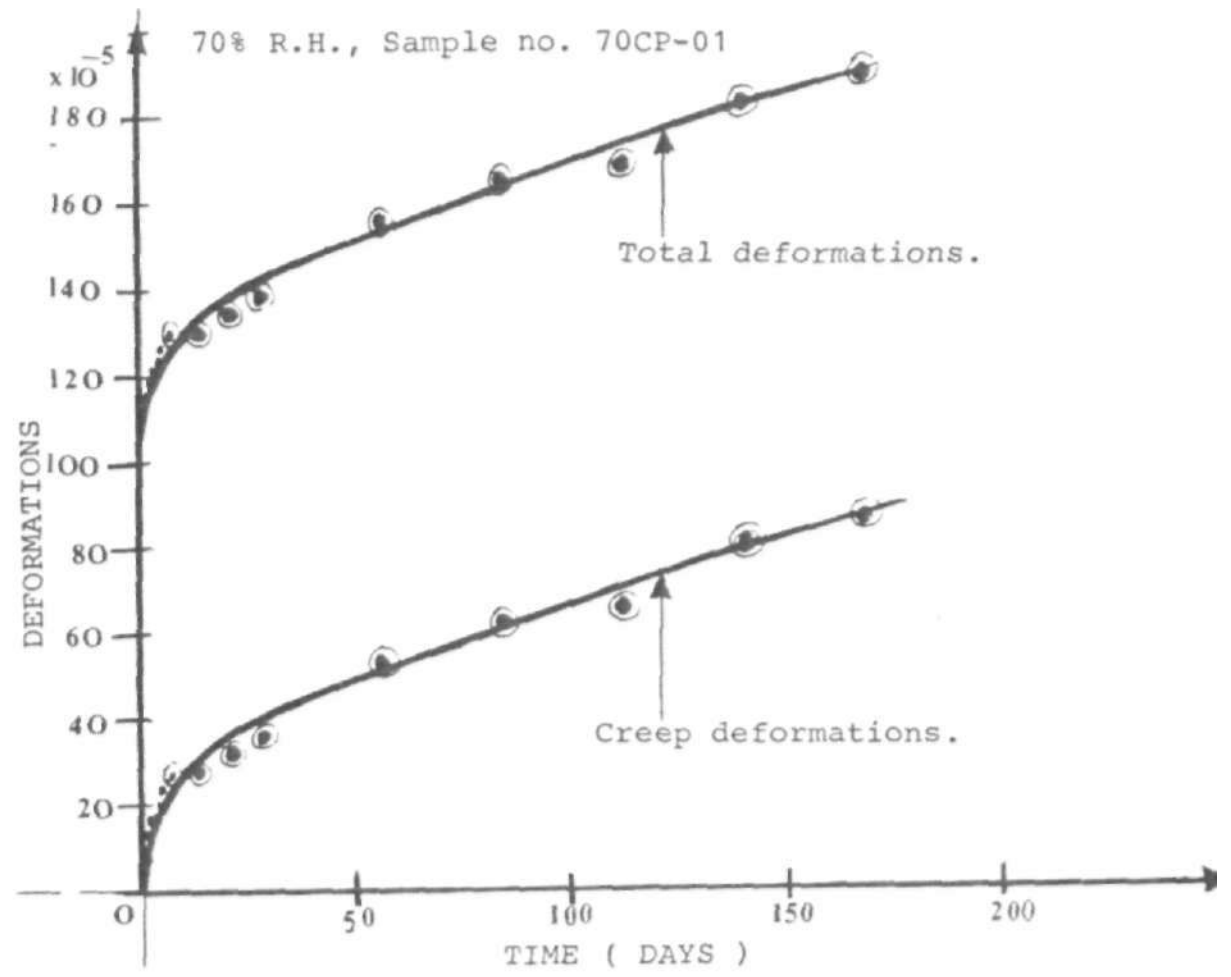


FIG.27. DEFORMATION AGAINST TIME.

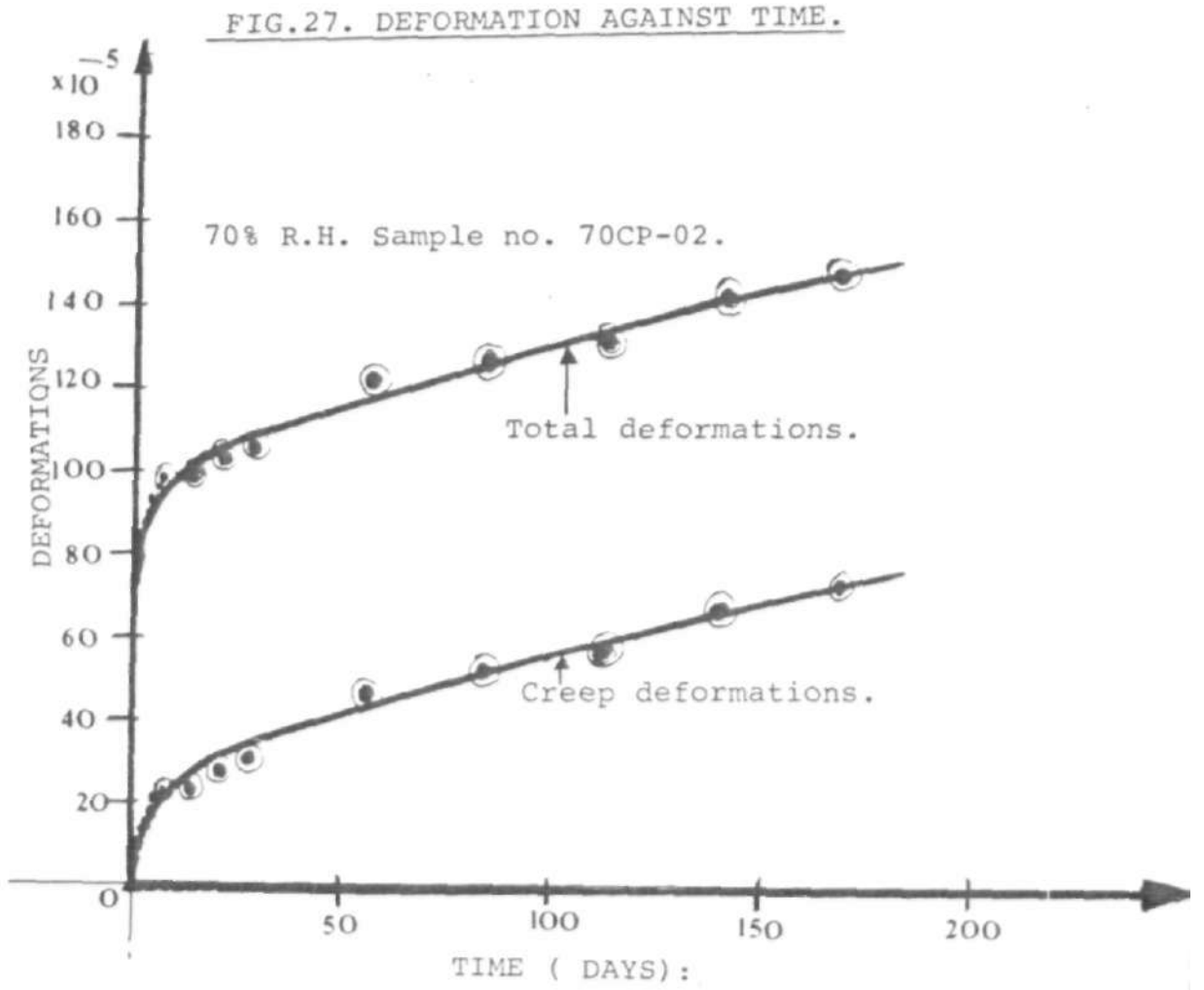


FIG. 28. DEFORMATION AGAINST TIME.

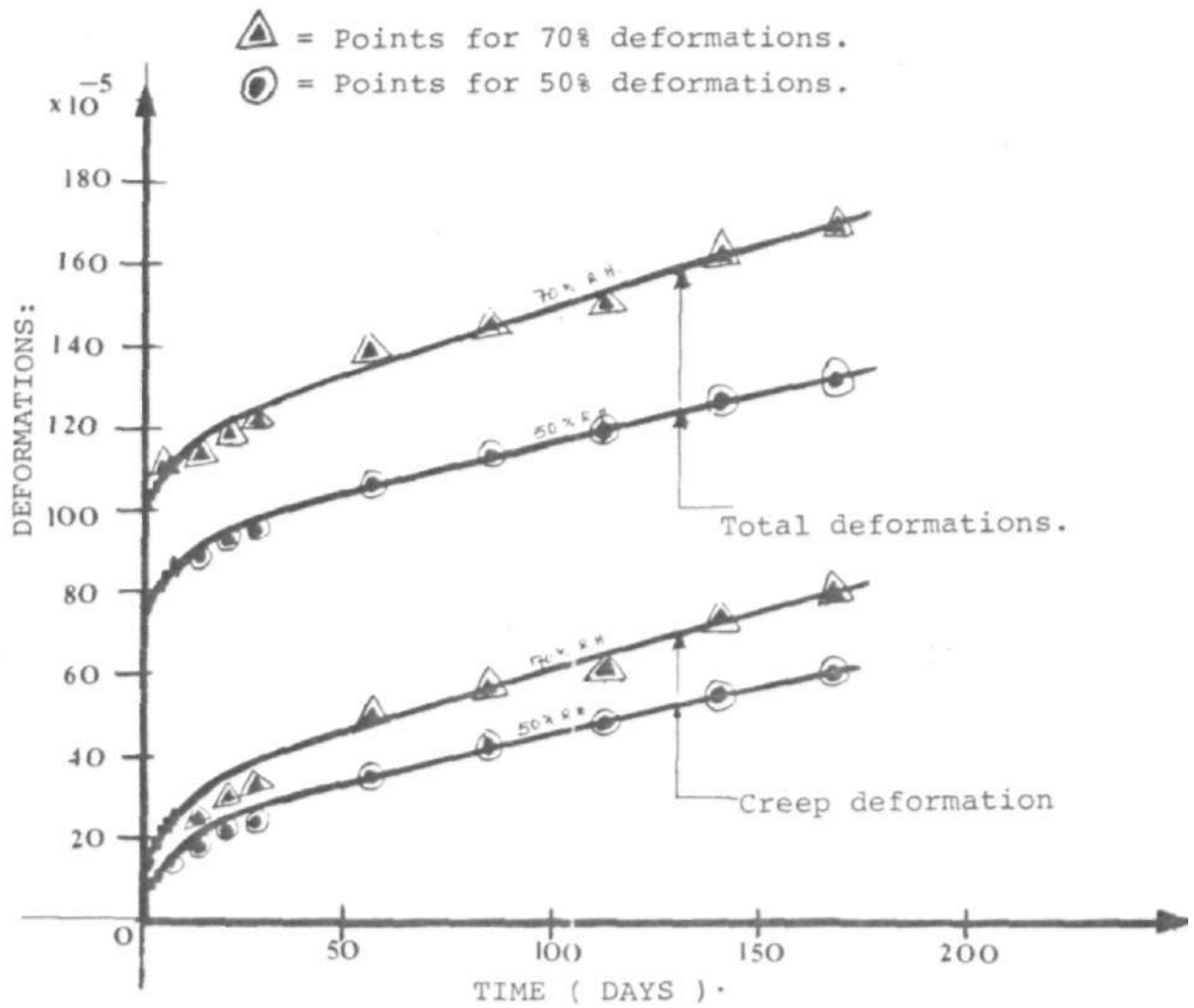


FIG.29. DEFORMATION AGAINST TIME.

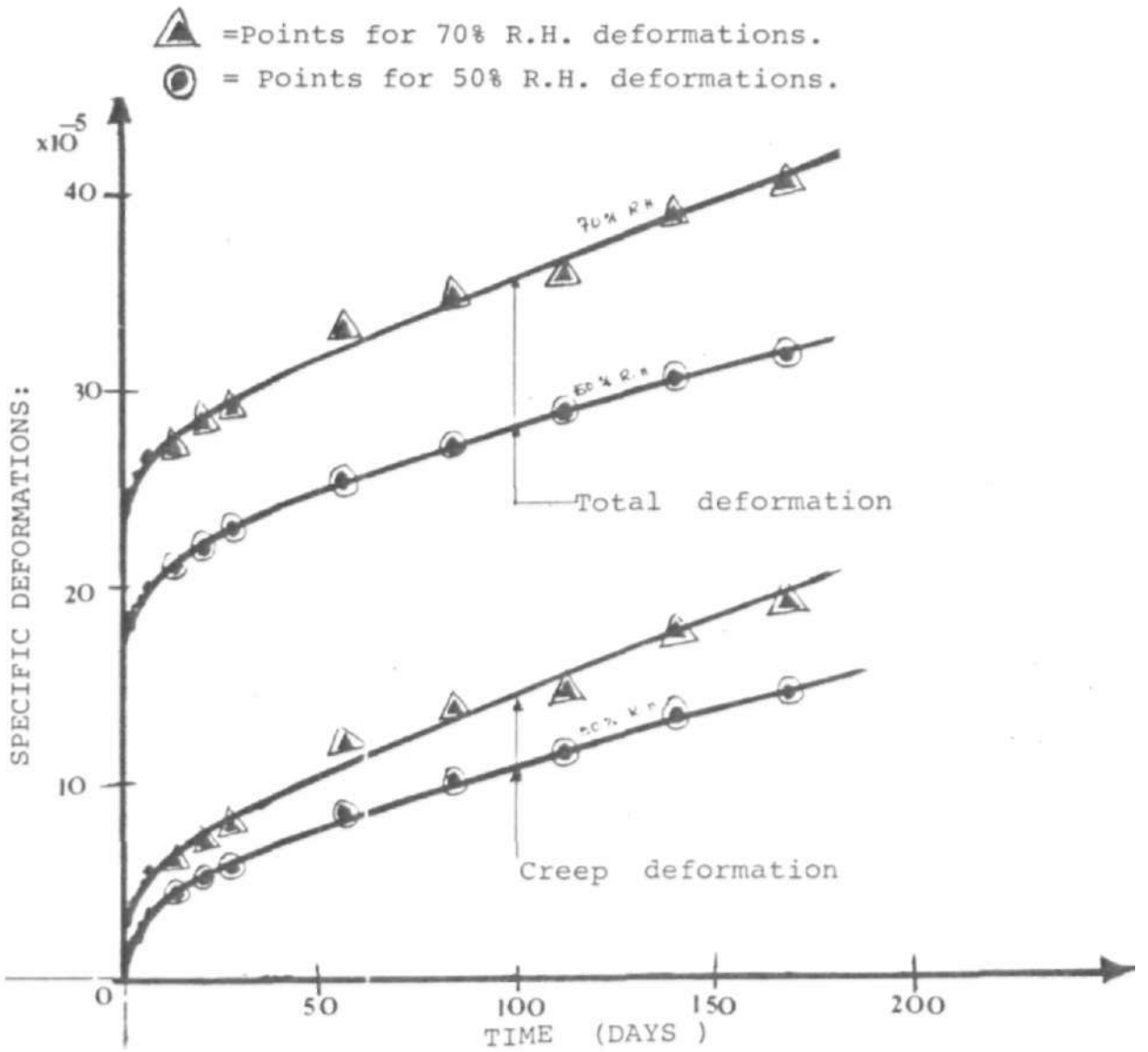


FIG.30. SPECIFIC DEFORMATION AGAINST TIME.

applied stress. The plot of the ratio of total deformation at time (t) in days to the immediate deformation on loading is also shown in figure 31. One observe that the deformation time curves are steep to begin with, as such considerable creep takes place in the first few days after application of load, the rate tending to decrease with longer time under load. This behaviour of soilcrete deformation time curves are in accordance with previous research work of deformation time curves for concrete materials. Thus the shape of the curves are the same both for soilcrete and concrete materials. It also implies that the mode of deformation with time is the same both in soilcrete and concrete materials but the end values of the deformations are not the same as soilcrete seems to deform more than the normal concrete. One observe also that the creep of soilcrete materials stored at a relative humidity of 70 per cent is higher than that stored at a humidity of 50 per cent. The same behaviour applies to the total deformation. On observing the graphs of figures 29, and 30 , one can see that the difference between the deformation values for storage at 70 and 50 per cent relative humidities is not constant for both total or creep deformations. Instead the difference in deformation values at these two storage conditions increases gradually with time so that the space between the graph lines for various humidity widens with time but at very slow rate.

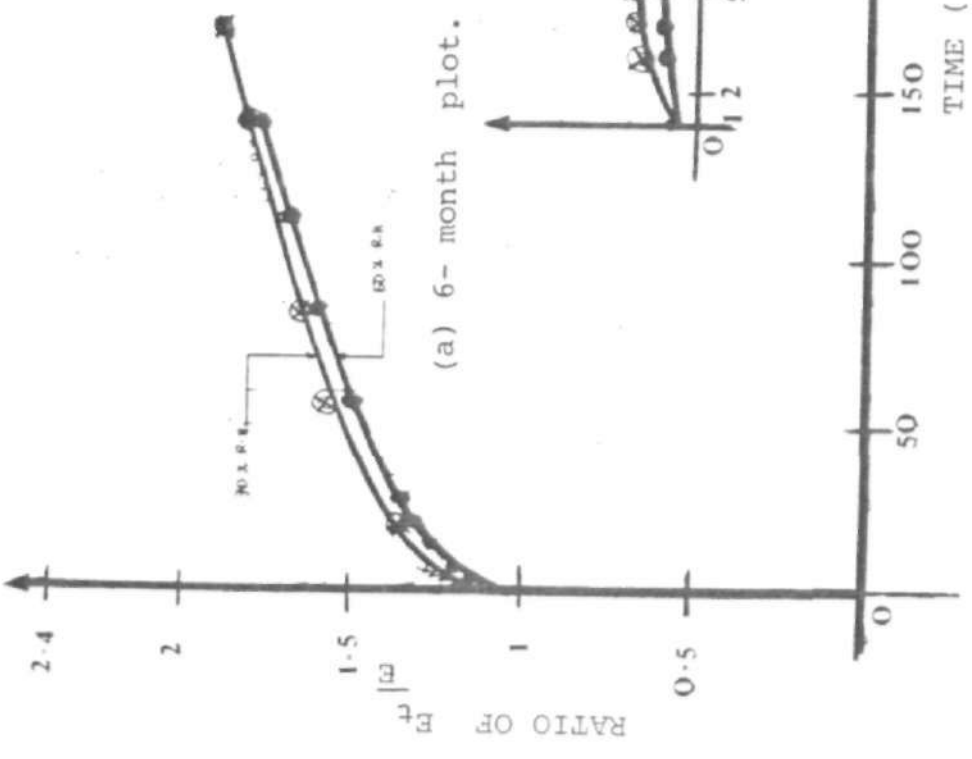


FIGURE 31. RATIO OF DEFORM. TO DEFORM. AFTER LOADING VS TIME.

Apart from the values of deformations being higher the higher the humidity, the rate of deformation is also higher the higher the humidity giving rise to the slowly increasing space between the graph lines for the various two humidities.

It is also interesting to not that the deformational response of concrete materials under various humidity is different from soilcrete material even though the mode of deformation and the shape of the graph are the same. In concrete materials the lower the relative humidity of storage, the higher the creep or total deformation because at a relative humidity of 50 per cent, creep deformation may be 2 to 3 times greater than at a relative humidity of 100 per cent as indicated in chapter two. This is because drying concrete creeps at a higher rate and achieves higher ultimate creep than concrete which remains wet or remains dry. However, the effect decreases with an increase in the size of the specimen. On considering, the influence of temperatue, concrete materials creeps more at higher temperature of storage. In other words, the higher the temperate, the more the deformation.

The soilcrete materials were tested at a relative humidity of 70 per cent and $14^{o}c$ for one storage condition and at a relative humidity of 50 per cent and $23^{o}c$ for the second storage condition. For concrete materials,

one would expect the storage condition of 50 per cent and 23°c to have higher creep deformations as this environmental conditions give rise to lower humidity and higher temperature compared to the other environmental condition. However for soilcrete materials, the reverse is the case as the creep and total deformations at a storage of 70 per cent humidity and 14°c is higher than that of 50 per cent and 23°c as indicated in the graphs. This behaviour of soilcrete materials may be due to the following reason.

1. the high content of the lateritic material in soil-crete (1 part cement to 12 part laterite) which absorb more water or moisture compared to hydrated cement, sand and aggregate in concrete.
2. The presence of water in the pores or capillary structure of the soilcrete materials giving rise to deformation response.
3. Soilcrete materials seem to be more plastic and contains more pores compared to concrete as indicated in section 3.3.0 of the thesis.

As one could see from section 3.2.4 of the thesis, the strength of soilcrete materials in water increases with time, as such the idea of more increase in humidity being dangerous is ruled out. This type of behaviour

are one of the complex problems facing most researchers in this area such that up till now no true complete mechanism or hypothesis explaining the nature of creep deformation have been established. However, one seriously believe that in the case of soilcrete material, higher humidity implies more water absorption into the pores or capillaries, such that it is the absorbed water responsible for the higher deformation, as compared to concrete with less pores or capillarry voids.

(a) Testing of Expressional Relations

In chapter two section 2.4.5 of this thesis, various basic expressions were given to predict the values of long-term deformation (creep). These expressions are basically for concrete materials and they are tested to see if they agree with soilcrete materials as well as concrete. These expressions have various constants and in order to obtain these constants various graphs using the soilcrete data were ploted. Some of the graphs are shown in figures 32 to 35. Other graphs plotted numbering about eight to ten were not shown since it was only the constant that were determined from these graphs.

On observing the ones shown (figures 32 to 35) which is mainly a plot of either natural or normal logarithm

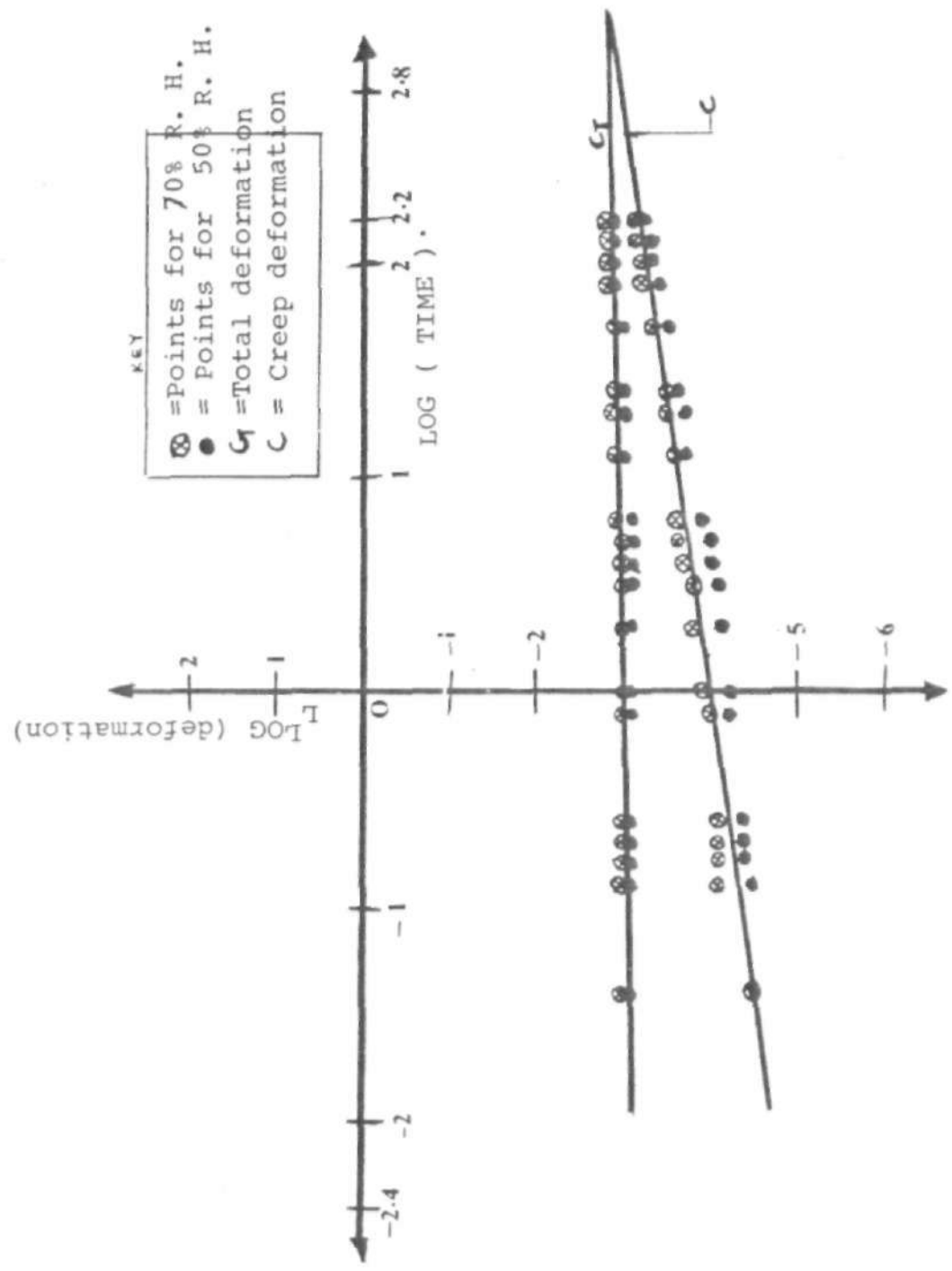


FIG.32. DEFORMATION AGAINST TIME. (LOGARITHMIC PLOT.)

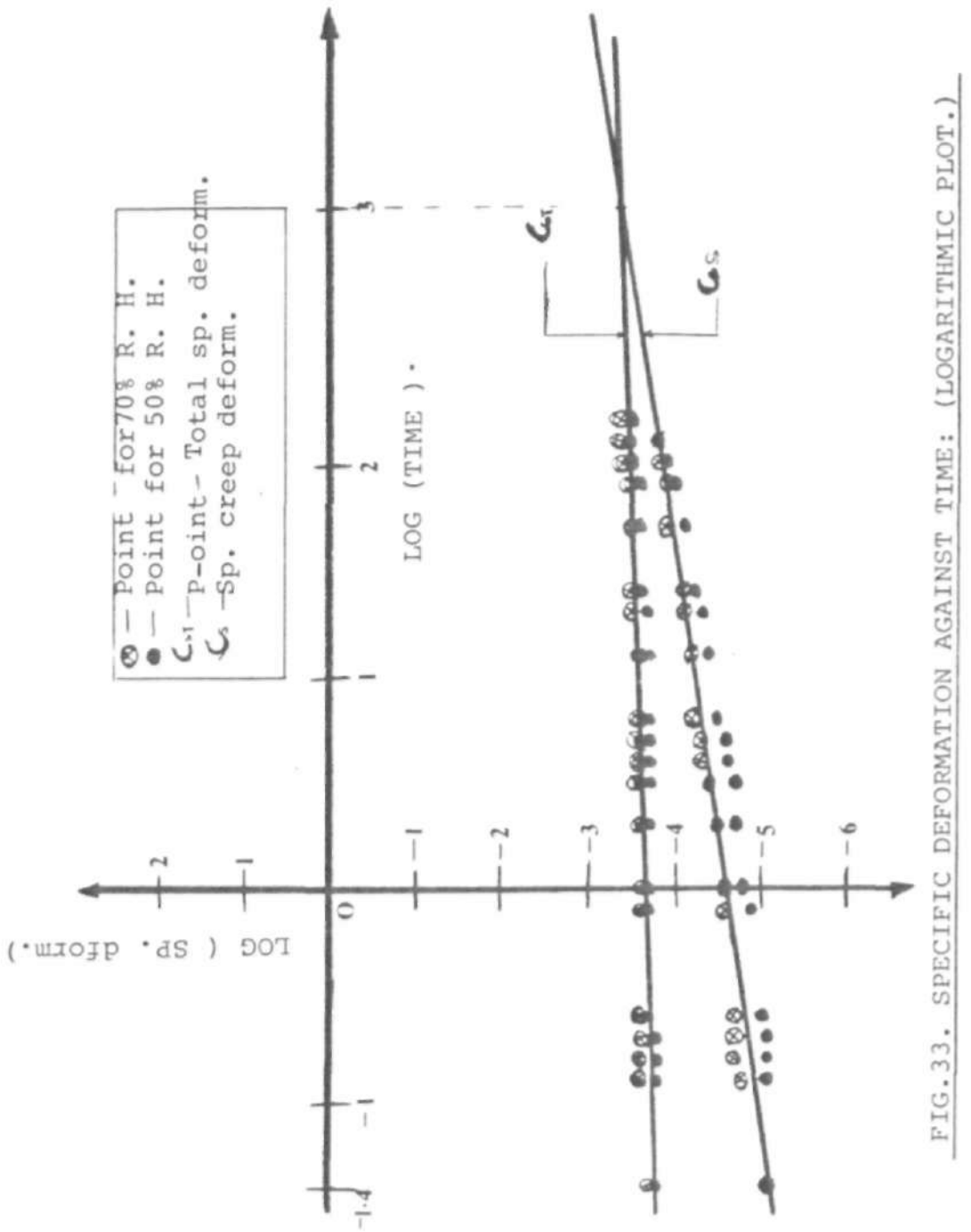


FIG.33. SPECIFIC DEFORMATION AGAINST TIME: (LOGARITHMIC PLOT.)

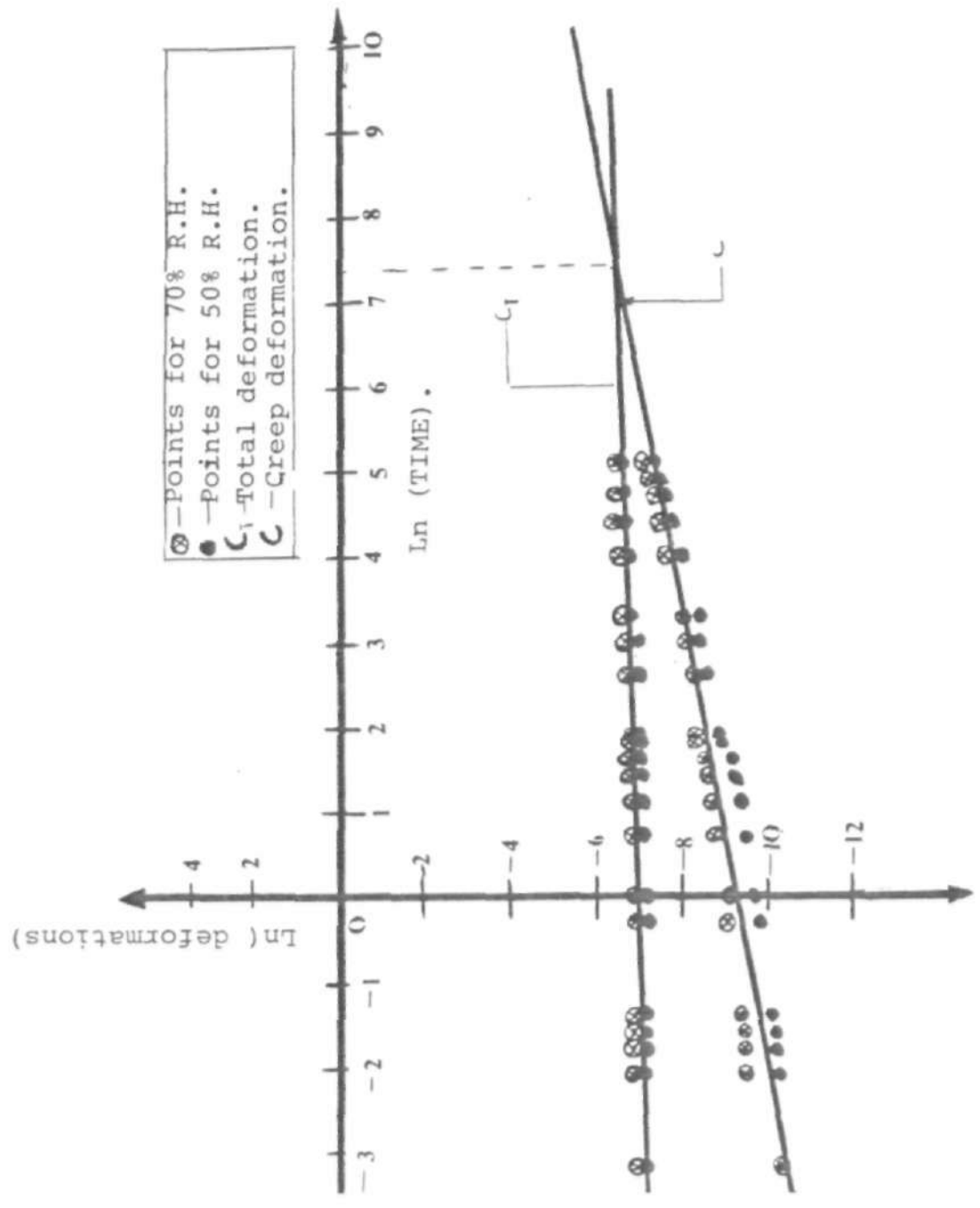


FIG.34. DEFORMATION AGAINST TIME: (NATURAL LOGARITHMIC PLOT).

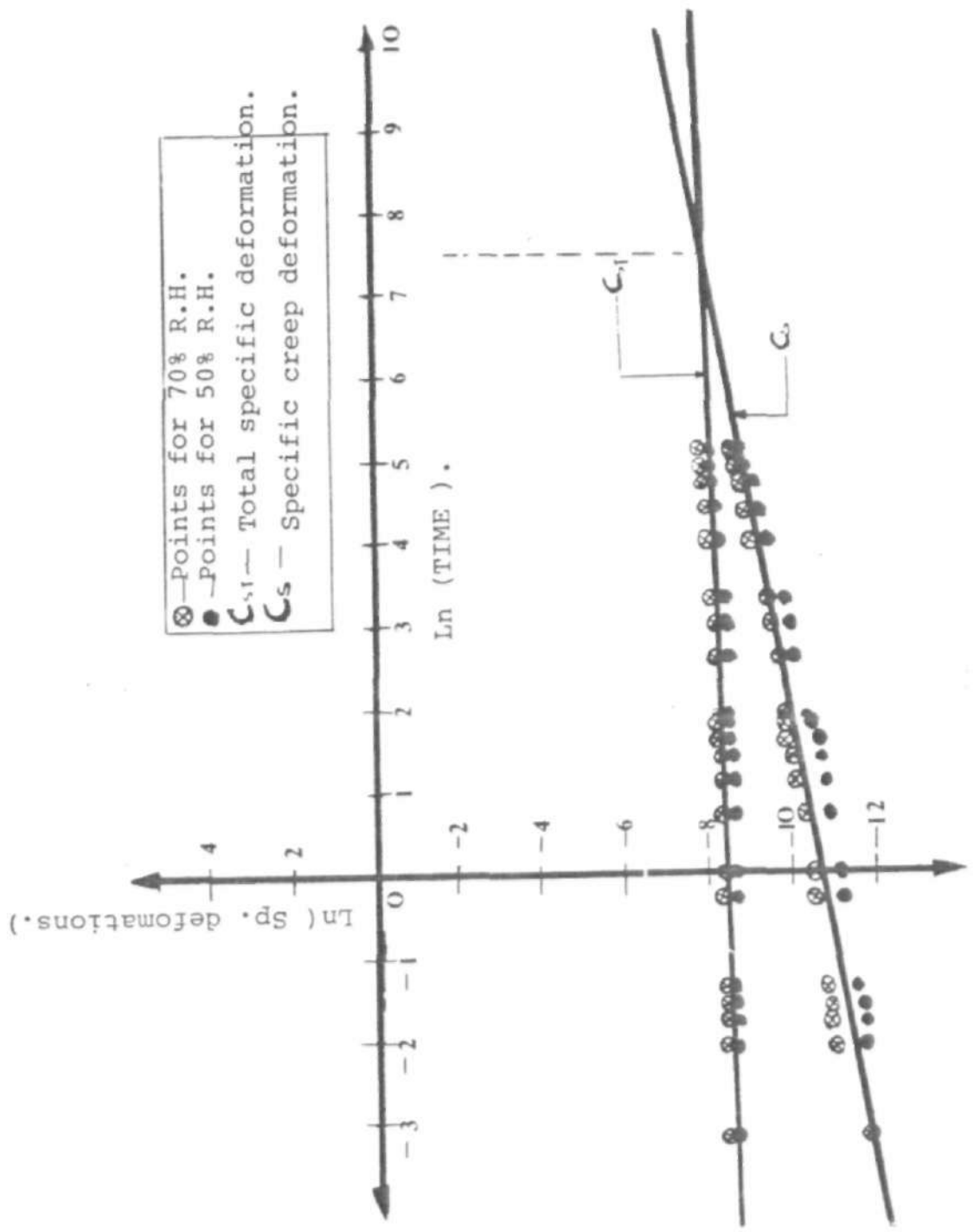


FIG.35. SPECIFIC DEFORMATION AGAINST TIME: (NATURAL LOGARITHMIC PLOT).

against natural or normal logarithm of time in days, one observe that in each case, at about 1000 days under load, the value of creep seems to be equal to the total deformation (point of intersection). One may regard this as the value of the limiting creep, though it is not measured in this thesis and would rather require more test for such a value to be established for soil-crete materials.

In testing each of the expressions, twenty points are chosen which are evenly spanned over the ten months periods and the residuals of each point determined with respect to the equation being tested. The resudual for each point is squared and the sum of the squared residual for the twenth points are determined. If the sum of the squared residual is a minimum, then the equation concerned is said to fit the soilcrete experimental data in the least square sense or in a least square parabola or hyperbola depending on the equation, and termed it a measure of the goodness of fit. On the other hand, if the sum of the squared residuals is not a minimum, then the opposite is the case and the equation concerned is said not to fit the experimental data for soilcrete and termed as not being in agreement with experimental data. In taking the residuals, only the absolute valued is

considered and squared, as squaring a value of say $2x\ 10^{-5}$ will be very small compared to when the absolute value of 2 is squared. It is also worth noting that the square of the absolute values are small since for example, in actual sense, the case of 2 above is 4×10^{-5} and not 4 when the deformation is considered. It is also neeessary to note that the actual square of 2×10^{-5} is 4×10^{-10} which is very small. Thus if one assumes a residual of absolute value of 2 for each of the twenty points, then the sum of the residuals is 80. Therefore in considering the value of the minimum which will determine whether or not any equation is in goodness of fit, 80 or less is chosen or regarded as a minimum. Thus in the proceeding pages, where the sum of the squared residuals is less or equal to 80, the equation concerned is said to be in agreement,otherwise, it is not. The values of the sum of squared residuals are not stated where the equation are not in agreement and stated where it is in agreement and deemed necessary.

(1) EXPONENTIAL EXPRESSION

These expressions are shown in table 3 of chapter two and include equations 20 to 25. In general, the exponential expressions do not agree with soilcrete experimental data. Thus some of the equations under-estimate the value of creep while others over-estimate the soilcrete creep deformations.

In equation **20** , there is need to measure the creep of soilcrete up to one year to enable one obtain the limiting creep and one observe that the values obtained as limiting in all the exponential expressions are not the actual limiting creep for soilcrete materials. Secondly the value of constants for the equations need to be establised and these require large experimental data for soilcrete.

McHenry /253/has made a lot of improvements on equation **21** , yet he observed that his predicted values does not agree with experimental data. Thus soilcrete is not an exception.

Equation **22** over-estimate the creep values within the first day under load and under estimate the creep values after one week or longer period under load for soilcrete materials.

Equation **23** cannot be truely checked since the shrinkage of the specimen is controlled and virtually zero. Thus there is no ultimate shrinkage and if applied the creep value will be zero. Since creep occurs in absence of shrinkage and shrinkage is not a condition or means by which creep occurs, the expression is not the best to be used. Secondly the expression is similar to the expressions in equation **21**

and 22 which does not agree with the soilcrete experimental data.

Equation 24 and 25 have their constant as imperical constant. Using the values of constant given, the expressions seem to under-estimate the values of creep for soilcrete materials.

In conclusion, all the exponential expressions can be said not to fit for soilcrete experimental data in the least square sense. Previous research work also indicate that these expressions do not have a good fit experimentally. One tend to attribute these poor measure of goodness of fit to the value of the limiting creep which is often seen in each of the expression. The values of the limiting creep obtained in each case seemed to be small in case of soilcrete as measured values within a short perid is very much higher than that calculated from the data using the expression. For such a limiting value of creep for soilcrete to be determined, it requires several years under load and large experimental data.

(2) POWER EXPRESSIONS

Generally the power expressions as shown in table 2 of chapter two agree very well with soilcrete experimental data except for one or two cases.

Table 2 of chapter two shows the power expression as equations 13 to 19 .

On testing equation 13 , it was observed that the quotient B is 0.385 and 1/B is 1/2.6. Comparison with measured and predicted values agrees very well and the sum of square residuals is $8.6 \simeq 9$. Thus equation 13 suggested by Straub /362/ is said to fit into soilcrete experimental data in the least square sense and the measure of goodness of fit is very good.

Equation 14 suggested by Shank /350 / is the same as that of equation 13 above when carefully observed. This is because the value of the stress in equation 13 is constant since only constant stress were applied to the specimens. Thus the value of constant A is 1. Equation 14 can therefore be said to agree with experimental data in the least square sense and the value of sum of residuals quoted above equally applies to this equation.

Equation 15 is an improvement on Shank's equation (equation 14) /350/ by Friedrich / 14 1/. Friedrich believed that after 1400 days under load the limiting creep would have occured or can occur any time from 1400 days under load. When logarithm of both side of the equation is taken and a graph of log C is plotted against log (t/1400), the slope should be 1/4 and the intercept should give log A. One observed that instead of the slope being 1/4, it is 1/3 for soilcrete materials. It is interesting to note that

when a power of 1/4 is used instead of 1/3, the predicted values does not agree with the experimental measured values. However, when a power of 1/3 is used which is the actual slope of the graph, the predicted values agrees with experimental measured values up to 60 - days under load for 70 per cent relative humidity storage only. Thus this improvement of introducing 1400 days as a time for limiting creep to occur may have suited a particular concrete as measured by Friedrich /141/. This improvement therefore does not make it any better for soilcrete rather it worsen the case except for 70 per cent relative humidity storage as indicated above.

Again in equation 16 suggested by Saliger /343/ one would expect the slope of a logarithm plot to be 1/3. On plotting the logarithm of both side of the equation, one observed that the slope is 1/2.6 (0.385) instead of 1/3 (0.3333) for soilcrete material. However, on comparing the predicted values with the measured experimental values, it is observed that both powers of 1/3 and 1/2.6 agree very well with experimental data. Even though the power (quotient) of 1/2.6 seems to be better fitted to the experiental data, one observe that the predicted values using the power of 1/3 are close to the measured values of 50 per cent relative humidity of storage and that of 1/2.6 to that of 70

per cent relative humidity of storage. Thus, a small change in the slope of the graph has a great effect on the predicted values. Thus equation **16** agrees very well with soilcrete experimental data in the least square sense and the goodness of fit is good. The same value of sum of residuals in the previous equations also apply here.

Equation **17** also suggested by Saliger/343/ who express the total deformation in terms of elastic deformation on loading, also agrees very well with soilcrete materials. On comparing the pridicted values with the experimental measured values, it is observed that the sum of residuals are 0.68 and 1.77 respectively for 50 and 70 per cent relative humidity of storage. Thus the equation fit into the soilcrete experimental data very well in a least square manner and the goodness of fit is excellent. However, it is worth noting that in Saliger's view /343/, the equation cannot be applied for loads applied at ages greater than 30 months. In the case of soilcrete, the load was applied at an age of 11 months.

This equation can be further modified to give effective modulus of the soilcrete raterials with time. Thus the effective modulus for various ages under load are determined and the graph is shown in figure 36 for

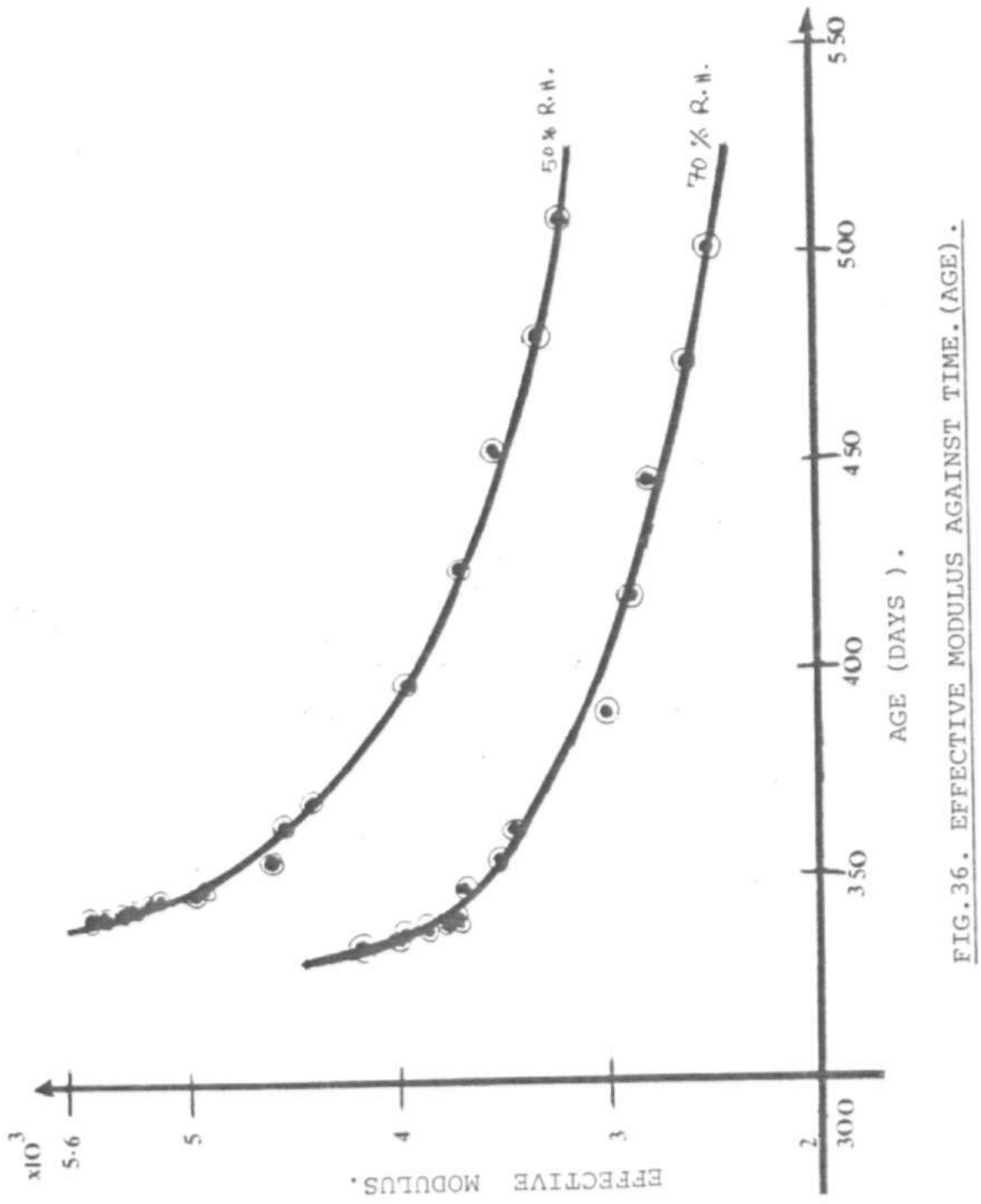


FIG.36. EFFECTIVE MODULUS AGAINST TIME.(AGE).

50 and 70 per cent relative humidity of stogage. One observe that the effective modulus decreases with time under load and tends to an assymptotic value with time. This is of course theoretically trues as the applied stress is constant and the deformation increases with time. It is also observed that the graph is non-linear, and the value of effective modulus for 50 per cent relative humidity is higher than that of 70 per cent relative humidity of storage. This could be as a result of more deformation observed in the case of 70 per cent relative humidity of storage.

The latest research publication on the effect of relative humidity on modulus of elasticity appeared in magazine of Concrete Research, Volume 41, Number 146 of March 1989. In this paper, AL-Mudiaheem investigated into the effect of ambient relatvie humidity on dynamic modulus of Portlant Cement paste and mortar. He concluded with the following, (1) the dynamic elastic modulus of paste and mortar drops with drying and attributed it to movement of water through the specimen. (2) The drop in dynamic modulus increases with increasing severerity of drying. This he attributed to the increase of stress gradients and removal of interlayer water caused by drying at such low relative humidities. His relative humidities

various from 0, 50 to 100 per cent. One observe that in soilcrete materials even though it is effective modulus and not dynamic modulus, the effect of relative humidity is different. The effective modulus at 50 per cent relative humidity is higher than 70 per cent relative humidity of storage. Rather than drop at 50 per cent, it increases when compared to 70 per cent. Since soilcrete materials are mainly lateritic in content and absorbs a lot of water; and the stress applied is constant, the reasons as given above by Al-Mudaiheem *for cement paste and mortar may* also be responsible for the opposite behaviour of deformation in soilcrete materials. At 70 per cent relative humidity, the soilcrete materials absorbs more water than 50 per cent and the interlayer water contribute partly to the total deformation thereby increasing the deformation and consequent reduction in effective modulus. At 50 per cent relative humidity significant amount of the interlayer water is lost and the hydration product is now responsible for deformation leading to less deformation as significant water is now lost with consequent increase in effective modulus.

Equation 18 proposed by Parrott /298/ does not agree with the soilcrete materials. The sum of residuals are over one thousand both for 50 and 70 per cent relative humidities. However it was observed that the equation agrees very well for both 50 and

70 per cent relative humidities of storage within the first month under load as the sum of residuals under this time of loading is less than 3 in both cases. Beyond this period under load, the expression does not fit into soilcrete experimental measured values in the least square sense. This may be due to the following possible reasons. The creep reduction g(t) which is a function of age at loading as determined from Parrot's graph for concrete materials may not be the possible value for soilcrete materials as such predicted values for longer periods under load does not fit into the measured data. The possible value of creep reduction function for soilcrete requires more tests to obtain large data and longer durations up to five to ten years under load.

One was most suprised and astonished as equation 19 proposed by Bazant et al /34, 35, 36, 39, 40, 42, 43, 44, 45, 46, 47, 51, 52, 53/ does not agree with soilcrete experimental data. Much hope was built on this equation (Equation 19) as Bazant et al has worked extensively on this area, though England and Illuston have always stress the point that the equation is more theoretical than practical. In most cases, Bazant et al usually give the range of constants involved and also the expressions to obtain the constants so that any environmental conditions could be applied. On trying both the various conditions and ranges of constants

involved, one observed that the equation over-estimate the creep values within the first two weeks under load and under-estimate the values for longer periods under load. The sum of residuals are large and is over four thousand in each case. This non-fit and lack of goodness of fit may be due to the following reasons.

(i) The assymptotic modulus used by Bazant et al in most of their expressions is obtained using the ACI and CEB expression for E-modulus. These E-modulus expressions are emperical and does not agree with the soilcrete materials E-modulus as discussed in section 3.3.5 of this thesis.

(2) A part from the value of E-modulus being higher than that for soilcrete when the expressions for ACI and CEB are used, this value is further multiplied by a factor of 1.5 to obtain the assymptotic modulus used in Bazant et al expressions. This further increases the value thus making equation 19 to over-predict the value of creep.

(3) There is also a limiting value of creep or creep reduction function in his expression. This limiting creep value often found in exponential expressions or creep reduction value also observed in Parrott (equation 18) in which the expressions do not agree with soilcrete experimental data seems to

have a different value for soilcrete materials. Thus the values so obtained using the respective expressions for soilcrete material is not true enough

In conclusion, except for few cases, the reasons given above, the power expressions agree very well with the soilcrete experimental data.

(3) HYPERBOLIC EXPRESSIONS

The hyperbolic expressions are shown in table 4 of chapter two and the equation number ranges from 26 to 29 . The hyperbolic expressions generally agree very well with soilcrete experimental data for period under load of less or equal to 60 days. Otherwise, the expressions can only have goodness of fit if the straight line in the plot of t/c, against t or 1/t is tangent to the point concerned.

Equation 26 was proposed by Ross and Lorman /240, 326/. On testing this equation, a plot of t/c against t was carried out to obtain the constant A and B. One expect that the points will lie on a straight line with intercept giving A and slope B. One observe from the graph that the points for 60 days under load fairly lie on a straight line, otherwise the graph is curved and non-linear. Thus, the expression agrees very well with experimental data for soilcrete for period under load of less or equal to 60 days. For longer periods

under load, the predicted values using this expression agree with measured values only if the straight line to determine the constant A and B in the plot of t/c against t is drawn tangent to the points or point concerned. Thus for each point or points in the a straight line, a value of A and B are obtained. Otherwise it does not agree with experimental data for soilcrete for period of more than 60 days under load. This is ofcourse tedious and time consuming. It is interesting and worth mentioning that the authors of this expression noticed this particular behaviour and termed it as varying in steps /286/ for concrete materials also. They also observed that the mean value of the ratio of constant A to B is 137 and allowing for discrepancies at early ages, they assumed this ratio to be 100. This led to equation 27 /326/

On testing equation 27 , it behaved exactly the same way as equation 26 . Though the predicted values using equation 26 seems to be fairly better for 60 or less days under load when compared with measured values. The rest remain the same. Thus the introduction of 100 into the expression has not improved it for the case of soilcrete materials. Again the value of 100 for a ratio of A to B may have suited a particular experimental data for a particular concrete concerned.

Equation 28 behaved exactly as the other two previous hyperbolic expressions discussed above. It predicts creep

fairly well under 60 days or less under load and under-estimate the value of creep for longer period under load with soilcrete materials.

For equation 29 , a plot of C against 1/t was used. It was observed that the curve is more non-linear than the previous ones under hyperbolic expression. The non-linearity is highly pronounced so that the expression does not agree with experimental measured values. Initially it over-estimate the value of creep for a week under load and under-estimate the Γ^{value} for longer periods under load.

In conclusion therefore, the hyperbolic expressions apart from equation 29 agree with experimental values for soilcrete materials for 60 days or less under load. The goodness of fit is said to be good under this period under load. Previous Reseachers have observed this facts about hyperbolic expressions. Neville/286/ in his conclusion on hyperbolic expression state that " It seems that if it is possible to make at least a 60 day creep test, the equation of Lorman and Ross give

satisfactory results. They are extremely easy to apply and have the advantage of quickly predicting the ultimate creep. If no tests can be made, the method of C.E.B. have to be used". Thus soilcrete materials is not an exception.

(4) LOGARITHM EXPRESSIONS

The only ologarithmic expression is that given by United State Bureau of Reclamation which is discuseed in section 2.4.5 (d) of chapter two as equation 30 .

Apart from little deviations after few hours of loading, the predicted values using this expression agrees very well with soilcrete measured experimental values. In most cases, all creep values deviate at initial stage of loading as a result of some elastic strains occuring so that this deviation at initial stage is normal. When predicted and measured values are compared, the sum of squares are 61 and 69 for 50 and 70 per cent relative humidity of storage respectively. Thus the logarithm expression is said to fit the soil-crete materials' experimental data in a least square sense and the goddness of fit is very good.

(5) INDIRECT POWER EXPRESSIONS

The indirect power expressions are shown in table 5 of chapter two and numbered as equation 31 to

33 . The indirect power expressions and especially equation 31 given by CEP-FIP agrees very well with soilcrete materials. However the rest does not agree with soilcrete materials because of the way their E-modulus are obtained. Otherwise they agree with soilcrete materials.

On testing equation 31 , all the factors are obtained using A=40 to calculate the creep coefficient. The creep coefficient is multiplied by the elastic deformation to obtain the actual creep. The factor β5 is recently discovered to depend on the thickness of the member with time. Thus a range of graph giving the factor was plotted covering 5 and 95 percentiles within a range of thickness of 20cm to 5cm. Thus a mean percentile is 10cm thickness of specimen. Soilcrete specimens used is of diameter 10.3cm cylinder. It was observed that the expression agrees very well with soilcrete experimental data. For 50 per cent relative humidty storage, the equation agrees very well with the measured data when the factor β5 is within 35 percentile while that of 70 per cent starage, when the factor is within 5 percentile zone. Calculated sum of residuals are 2 and 3 for 50 and 70 per cent relative humidity. One would have expected soilcrete measured creep values to have the factor within the mean percentile or near mean percentile as the diameter of

soilcrete is 10.3 cm and not 7 cm. However this was not the case as the factor β_5 is within 35 and 5 percentiles for 50 and 70 per cent relative humidity. This could be due to the following reasons.

(1) Soilcrete creep more than concrete, as such the factor for age at loading which now depends also on thickness of specimen may not be exactly the same as concrete.

(2) The soilcrete specimens were loaded at an age of 320 days instead of the normal 28 days age of loading

(3) The relative humidity of storage may have played some role though these factors were taken care of in other factors involved in the xpression.

One interesting thing to note however is the fact that all the values irrespective of relative humidity of storage fall within the range of 5 and 95 percentile specified by CEB - FIP. Thus equation 31 agrees very well with experimental data in a least square sense.

Equation 32 proposed by ACI committee 209 like Bazant et al equations do not agree with soilcrete experimental data. It over-predict the creep values at the initial period (1st month) under load and under-predict creep values at longer period under load.

This may be due to the follcwing possible reasons:

(1) The ultimate creep coefficient, C_u, which is the ratio of the limiting creep to the initial strain at loading given as 2.35 for concrete may not be as well correct for soilcrete materials. Again the value for soilcrete require more tests, data and test duration up to five years to obtain an accurate ratio.

(2) C_1, C_2 are correction factors for relative humidity and thickness of member under test for the same ultimate creep coefficient. These need thorough cross-checking for soilcrete materials as that of concrete may not be necessary true for soilcrete.

(3) $(t)^{-0.118}$ is also a correction factor for age at loading for ultimate creep coefficient which again may not be correct for soilcrete materials.

(4) The value V_t is obtained using an expression similar to that of Bazant et al which do not agree with soilcrete experimental data.

Equation 33 was proposed by Deutsches Institut fuer Normen (DIN 4227) which is a German Code for prestressed concrete. Observing this equation carefully,

one can see that it is exactly the same as equation 31 proposed by CEB-FIP which agrees with the soilcrete experimental data. In both cases, it is elastic deformation multiplied by creep coefficient. However, in DIN 4227 paragraph 8.3, it is strickly specified that in obtaining the elastic deformation, the constant stress should be divide by the E-modulus obtained from table 6 of paragraph 7.3 of the same code. Secondly the ultimate creep coefficient is to be obtained from table 8 under paragraph 8.4 of the same code. The value for ultimate creep coefficient are 2 and 2.7 for 70 and 50 per cent relative humidity of storage. Thus using the expression as directed by DIN 4227 lead to equation 33 not agreeing with soilcrete experimental data because of the following seasons (1) The E-modulus obtained from table 6 are calculated using emperical form similar to those given in section 3.3.5 which does not agree with soilcrete E-modulus. The values are higher than that of soilcrete. (2) Again, the ultimate creep coefficient given for 70 and 50 per cent relative humidity may not be correct for soicrete materials as soilcrete creep more at 70 per cent relative humidity than 50 per cent. However if the E-modulus in the table is neglected and the immediate deformation after loading is taken with all other factors obtained from graph, the equation is the same with equation 31

and agrees with soilcrete experimental data in a least square sense.

(6) EXPRESSIONS FOR CREEP UNDER VARIABLE STRESS

These expressions are given in chapter two section 2.4.5 (f) as equation numbers 34 and 35 . In actual sense these equation are not checked as the applied stress is constant and not variable.

However, equation 34 which is the effective modulus method is exactly the same as shown in figure 36. This figure shows the variation of modulus with time under load. It should be noted that the applied stress is constant in this case.

In using eauation 35 , an expression for specific creep is obtained. This expression is divided by a time interval t to obtain $\partial C_{sp}/\partial t$. This is then intergrated with respect to time to obtain creep using equation 35 . As earlier noted, since the stress is constant, the expression cannot be tested.

The plot of the ratio of creep deformation at 70 per cent to 50 percent relative humidity of storage is as shown in figure 37. One can observe that the ratio tends to the value of one with time.

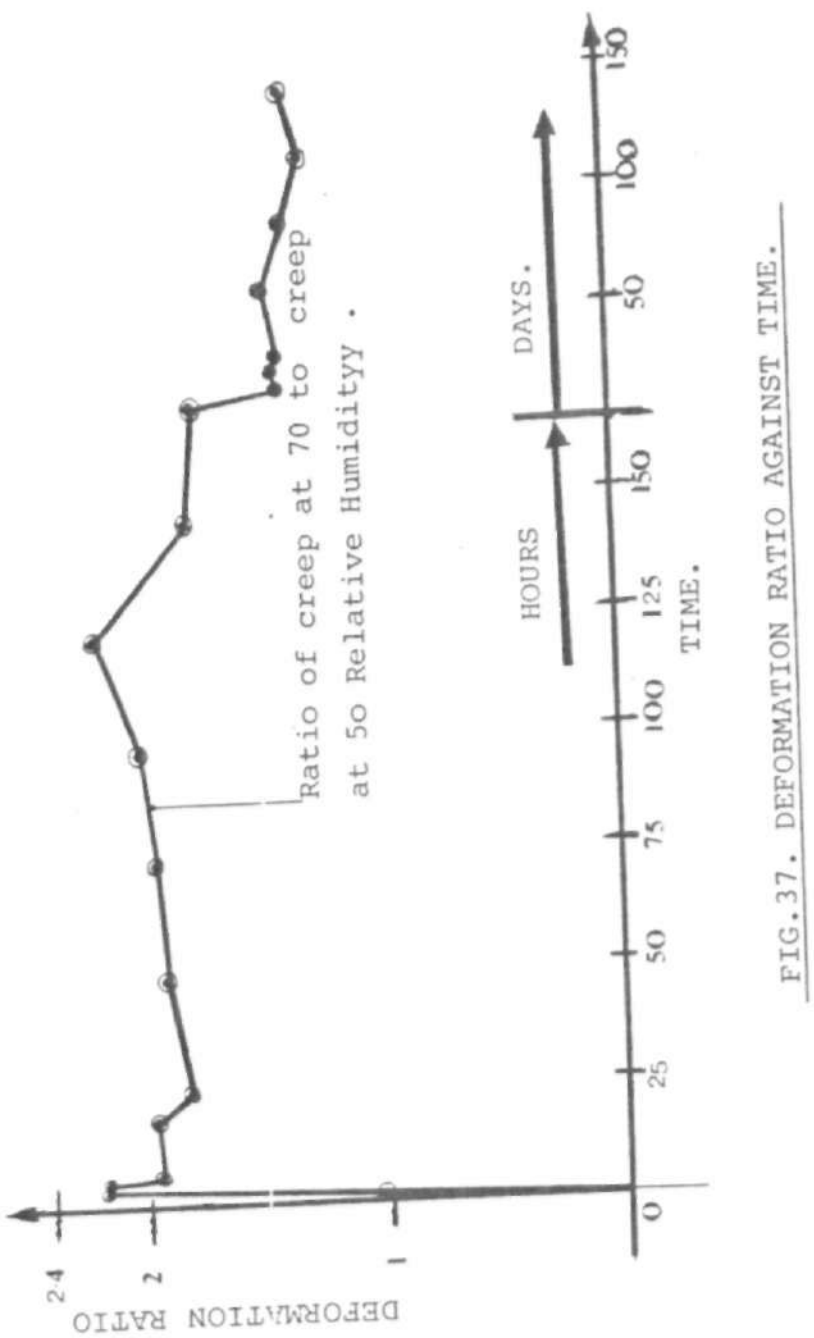


FIG.37. DEFORMATION RATIO AGAINST TIME.

3.4.6 OBSERVATIONS

From the long-term tests performed on the soilcrete materials and the analyses of test results that followed, the following observations can be made.

(1) The performance of non-stabilised oil hydraulic system of apparatus for creep deformation measurement is not quite satisfactory as it cannot maintain a constant load with time. This can be seen from the first loading which was discontinued as a result of this problem. Previous research works have also indicated this problem

(2) The stabilised hydraulic and spring loaded systems of apparatus for creep measurement have a very good performance and can be relied upon for longer periods of tests.

(3) The deformation time curves for soicrete materials are steep to begin with, giving rise to considerable creep taken place within the first few days, followed by the rate of creep tending to decrease with longer time under load. This behaviour is exactly the same with concrete materials. Thus the shape of the curves and the mode of deformation with time is the same for both concrete and soilcrete materials.

(4) In soilcrete materials, the creep or total deformations with time under load is higher at 70 per cent relative humidity than 50 per cent relative humidity of storage. These can be seen from figures 29 to 30. Thus the higher the humidity,the higher the deformation. However, one expect a saturation humidity where further increase in humidity will mean no further increase in deformation as soilcrete materials increase its strength with time when soaked in water. In concrete materials, the reverse is the case as lower humidity means higher deformations. This behaviour of soilcrete materials may well be responsible for the heavy pot-holes found in most Nigerian roads after one to three years of construction. This is because most Nigerian roads uses stabilised laterite or laterite as their sub-grade and one observe that despite good compaction and construction, the roads develope after one to three years of construction heavy deep pot-holes. Most of the pot-holes and depression often develope during raining season. With the knowledge of the soilcrete deformational behaviour with time, this problem can easily be arrested.

(5) The rate of creep or total deformation is higher the higher the humidity but decreases gradually with time. This can be seen from the fact that the gap

between the graphs of the two various humidities increases slowly with time under load.

(6) It is also observed that at about 1000 days under load, the value of creep deformation will be equal or near the value of total deformation for soil-crete materials. This can be seen from the logarithm plots shown in figures 32 to 35. However, this statement is to be verified experimentally as creep up to 1000 days under load for soilcrete is not measured in this thesis.

(7) The exponential expressions for predicting creep deformations do not agree with soilcrete experimental data as most of the expressions have poor goodness of fit and do not fit the data in the least square sense. This may be attributed to the limiting creep value which one think is small for soilcrete materials as measured creep values within short time is much higher than the value of limiting creep calculated. The need to establish that of soilcrete materials cannot be over-emphasized.

(8) The power expressions for creep deformation agree very well with soilcrete experimental data. The equations are said to fit the soilcrete experimental data in a least square sense. The sum of squared

residuals is about 9 which is far less than 80 in each case. However, equations 18 and 19 proposed by Parrott and Bazant et al did not agree with soilcrete materials.

(9) The hyperbolic expressions for creep deformations agrees very well with soilcrete materials' experimental data for period of 60 days or less under load. Previous research works have also indicated this type of behaviour for hyperbolic expressions.

(10) The only logarithmic expression tested agrees very well with soilcrete materials creep deformation values in a least square sense and the goodness of fit can be said to be very good.

(11) The indirect power expressions which are mainly expressions with factors that are obtained from graphs agrees very well with soilcrete experimental data. They are said to fit the experimental data in a least square sense especially when the elastic deformation is taken as deformation after loading and not by the uee of E-modulus obtained from the tables. However the expression as proposed by ACI Committee using Branson et al does not agree with soilcrete materials experimental data because of the

way the ultimate creep coefficient is calculated or assumed.

(12) Apart from exponential expressions the various other expressions employed for concrete creep deformations can as well be employed to soilcrete material deformation. Even the exponential expressions can also be used for concrete as well as soilcrete materials so long as the limiting creep values for soilcrete materials have been determined experimentally. Thus, all creep deformational expressions for concrete materials can also be equally used for soilcrete materials' creep deformation.

CHAPTER FOUR

CONCLUSION AND RECOMMENDATIONS

As with any research work, it will always end with the findings and recommendations of areas to be further worked upon. These areas are usually those that have not been thoroughly dealt with within the report. Thus this chapter is fully devoted to findings of the research work and vital areas that need urgent research attention.

4.1.0 CONCLUSIONS

From the extensive literature review coupled with the various experimentations, analyses and observations carried out in this thesis, the following conclusions can be ma . It should be noted however that in concluding, only the findings are stated. For detail discussions and justification of findings, one is reffered to the experimental result analyses and observations in the various sections of the thesis. The conclusions are based purely on the tests carried out on the laterite, stabilized soil known as soilcrete materials.

(1) No complete or sound mechanism or hypothesis has yet been established to fully explain the true nature of creep deformation.

(2) The lateritic materials which is of A-6 plastic clay soil classification can be used in the production of soilcrete materials. This classification is of the United State Bureau of Public Roads.

(3) Soft boards or celotex as packing materials during compressive strength test for soilcrete hollow blocks will distribute the load uniformly over the entire block's bed faces than any other packing material.

(4) Soilcrete hollow blocks develop about 90 per cent of its 28-days strength within one week of age and continues to increase its strength with time.

(5) The wet compressive strength of soilcrete hollow block is about 65 per cent of the 28-days dry compressive strength.

(6) The soilcrete hollow blocks have a critical soaking period of 1 day in water. The wet strength of the critical soaking period is about 40 per cent of the 28-day dry compressive strength.

(7) The best curing for soilcrete material is by soaking for 2 days in water after 24 hours of age. It is then left in the normal laboratory atmosphere for 28 days of age before testing.

(8) In soilcrete, the concept of water-cement ratio is not suitable, rather water-aggregate ratio is used.

The best water-aggregate ratio for soilcrete production is 0.200.

(9) The most suitable compaction pressure for soilcrete prod—uction is 9.0 N/mm^2.

(10) The average cylindrical compressive strength at an age of 320 days for soilcrete materials is 14 N/mm^2.

(11) The soilcrete materials have an average E-modulus of 5387 N/mm^2 tested under 65 per cent relative humidity and $20^{o}c$.

(12) An average ultimate strains under the above environmental conditions for soilcrete materials is 7.15×10^{-3}

(13) The soilcrete materials have an average dynamic modulus of 9122 N/mm^2.

(14) Most of the emperical relations as given by various investigators for determing E-modulus using density and compressive strength do not agree with soilcrete materials. More tests are therefore required to establish such for soil-crete materials.

(15) All the general expressions for the stress-strain curve envelope for concrete materials and light-weight concrete materials agree perfectly with the soilcrete materials.

(16) The soilcrete materials have the lowest ultimate strain as its crushing strength approaches zero of 4 $x10^{-3}$ (400 x 10^{-6}).

(17) The soilcrete materials have the constant modulus of ultimate deformation increament of 4850 N/mm^2.

(18) In both soilcrete and concrete materials the deformation time curves are steep to begin with followed by a gentle and gradual slope with longer time under load.

(19) In soilcrete materials, the creep or total deformations with time under load is higher at 70 per cent than at 50 per cent relative humidity of storage. The reverse is the case with concrete materials. This behaviour in soilcrete materials may be responsible for the heavy pot-holes found in most Nigeria roads.

(20) The rate of creep or total deformation is higher the higher the humidity but decreases gradually with time under load.

(21) At about 1000 days under load, the value of creep deformation will equal or near the value of total deformation for soilcrete materials.

(22) The exponential expressions for predicting creep deformations do not agree with soilcrete experimental data.

(23) The power expressions, hyperbolic expressions, logarithmic and indirect power expressions for predicting creep agree very well with soilcrete experimental data. They are said to fit the data in a least square sense and the goodness of fit is excellent.

(24) All mathematical expressions used for predicting creep of concrete materials can also be used for soilcrete materials except the exponential expressions which need further modifications.

4.2.0 RECOMMENDATIONS

The following areas are recommended for further research work.

1. Since no complete acceptable mechanism or hypothesis has been established to fully explain the true nature of creep deformations, one suggests a full creep study of material with known

molecular structure at the microscopic level. Such material like wood can be used for study of creep. Perhaps from this study, one can fully establish mechanism or hypothesis that can explain creep.

2. More tests on soilcrete materials should still be continued in area of E-modulus to enable an emperical expression for soilcrete to be established.

3. Tests should be carried on to determine the limiting value of creep for soilcrete materials. This implies a creep tests up to 5 - 10 years under load.

4. More creep tests should be continue on soilcrete materials to enhance the findings of this study and to further determine the critical relative humidity if there is any so that with the known and established deformations in soilcrete, the problem of pot-holes in Nigeria roads should be tackled once and for all.

CHAPTER FIVE

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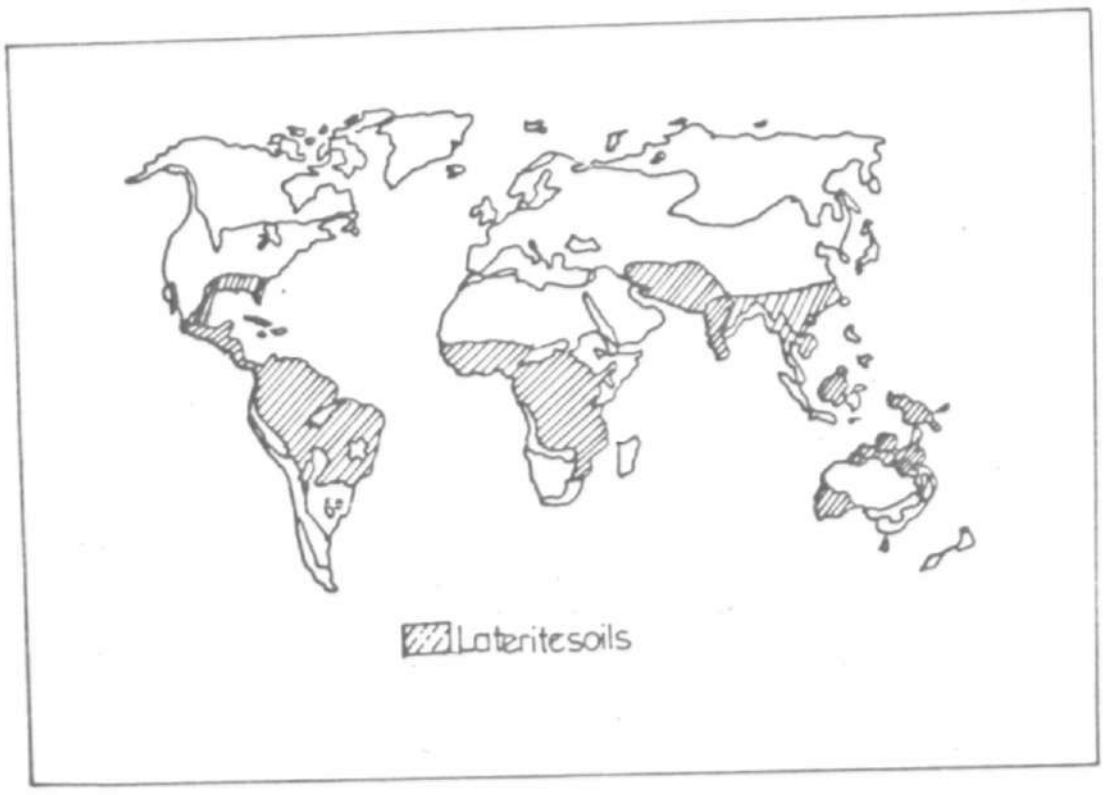
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CHAPTER SIX

APPENDICES

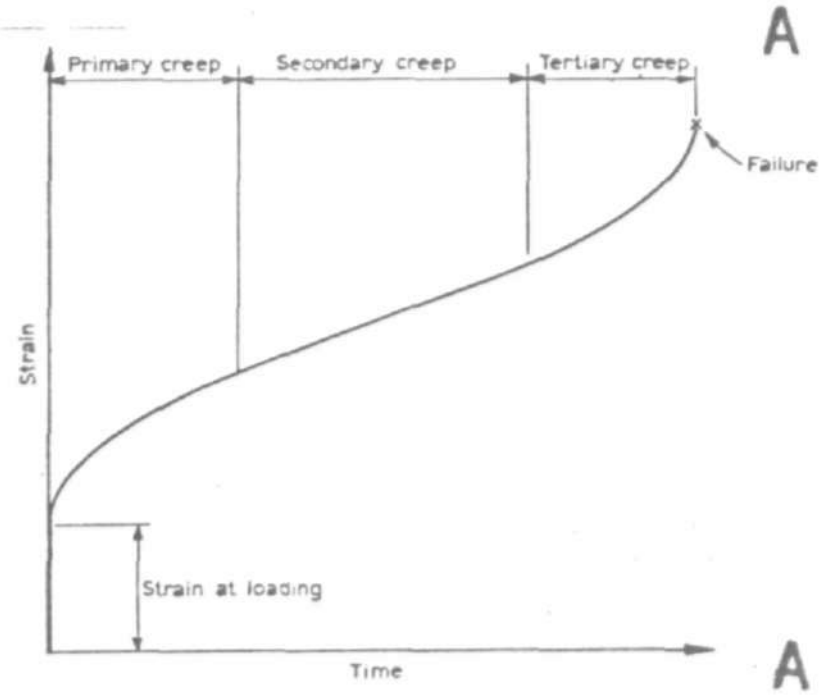


Generalized world map showing the distribution of Laterite soils

A 1 /2,153/

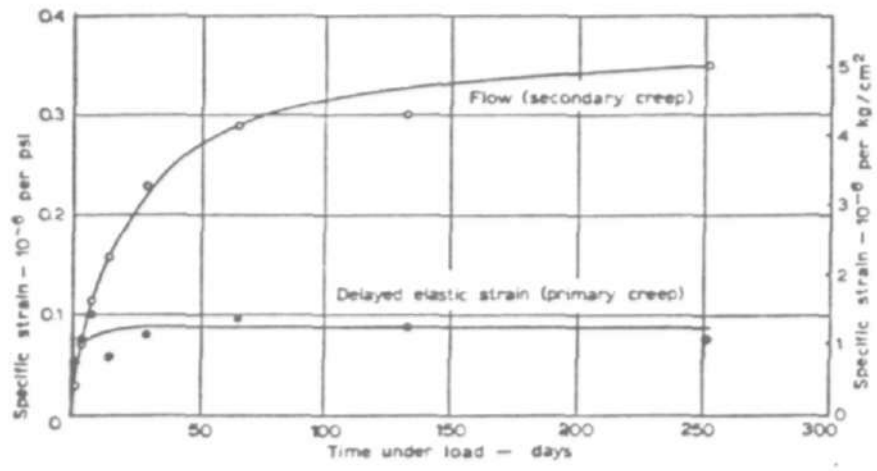
Initial stress in steel		Centre deflection immediately after loading		Deflection after 47 days of sustained loading	
psi	kg/cm²	in.	mm	in.	mm
3000	210	0.04	1.0	0.10	2.5
8000	560	0.10	2.5	0.17	4.3
16000	1120	0.15	3.8	0.29	7.4
29700	2100	0.20	5.1	0.37	9.4

A 2 /188,286/



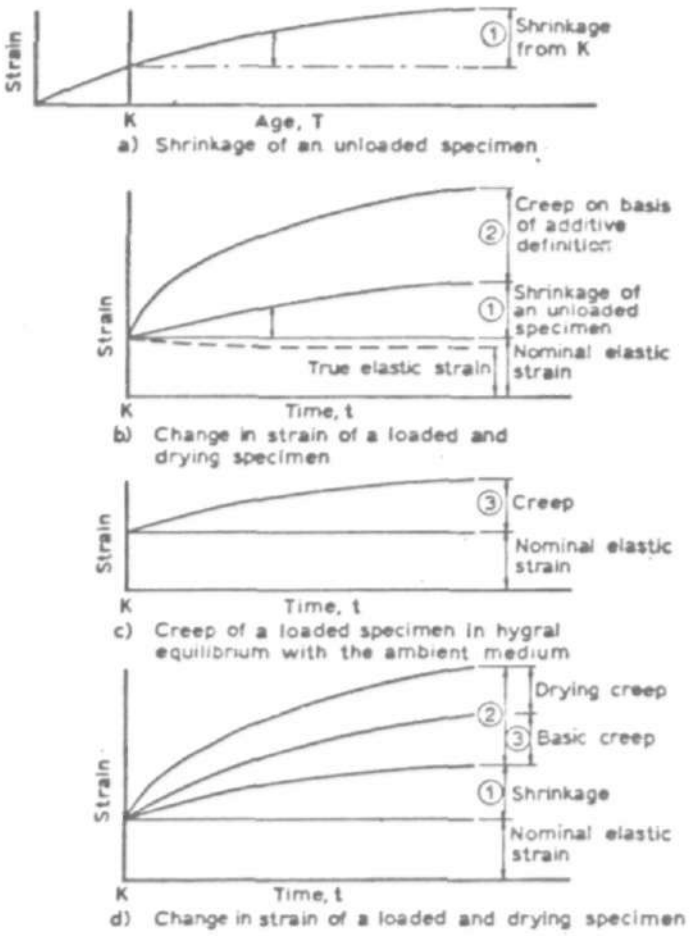
A 3 /286/

General form of strain–time curve for a material subject to creep.



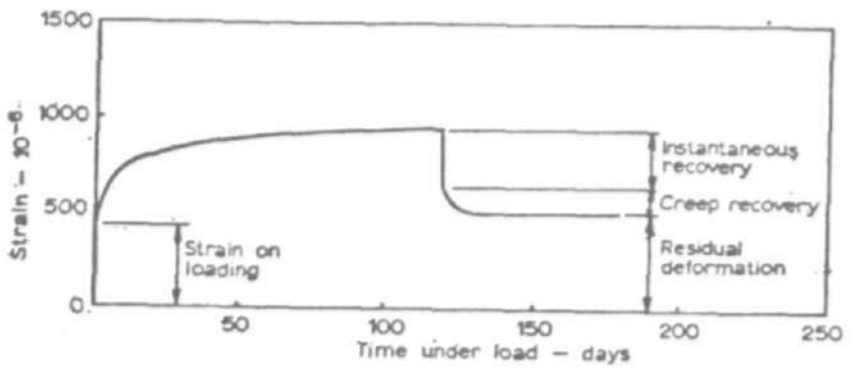
Primary and secondary creep for a 1:6.5 concrete with a water-cement ratio of 0.53.

A 4 /203/



. Definition of terms.

shows the terms and definitions involved.



Instantaneous and creep recoveries.

A 6 /286/

Secondary creep – verification of viscous flow conditions

Age at loading (days)	10	24	52	108
Observed secondary creep (10^{-8} per psi)	19.3	7.5	4.0	3.1
Deduced secondary creep from a specimen loaded at an *earlier* age (10^{-8} *per psi*)	24.0	11.1	4.3	1.6

A 7 /203/

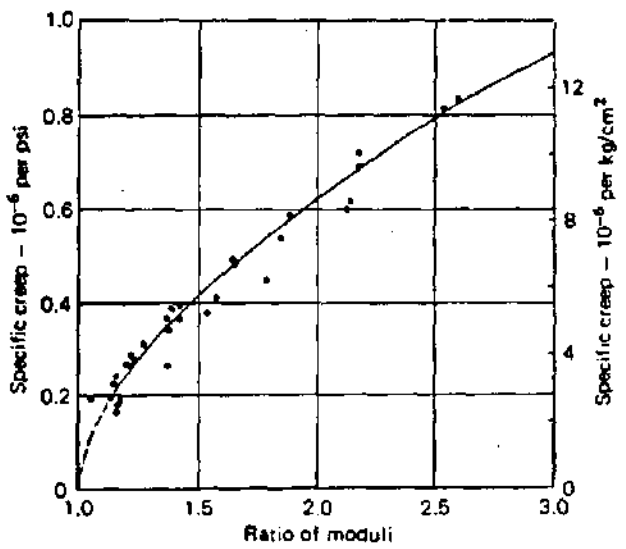
The German code DIN 4227 published in 1953 gives the following values of the ultimate creep coefficient ϕ_∞ for different conditions of storage of concrete:

in water	0.5 to 1.0
in very humid air, e.g. immediately above water	1.5 to 2.0
in open air	2.0 to 3.0
in dry air, e.g. indoors	2.5 to 4.0.

The value of ϕ_∞ depends also on the degree of hardening of concrete at application of load (ratio of strength at loading to ultimate strength), so that the values above should be multiplied by a factor k given below

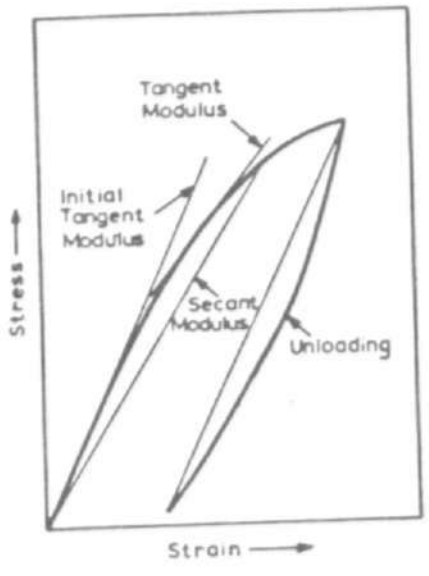
Ratio of strength at loading to ultimate strength	k
0.65	1.5
0.75	1.0
0.85	0.75
1.0	0.5

A 8 /116/



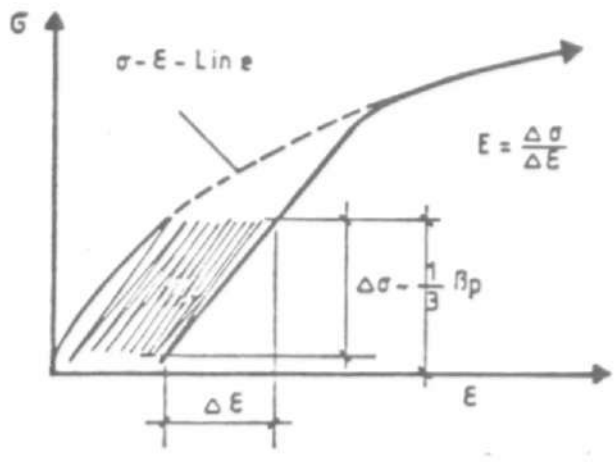
Relation between specific creep at any time and the ratio of the modulus of elasticity at that time to the modulus at the time of application of load

A 9 / 383/

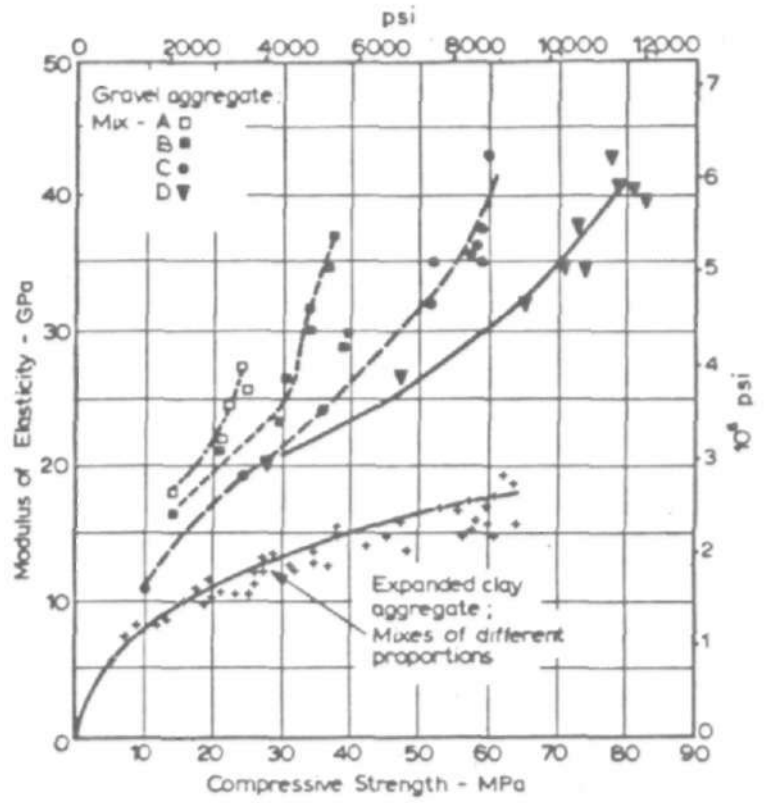


Typical stress–strain curve for concrete

A 10 /280/

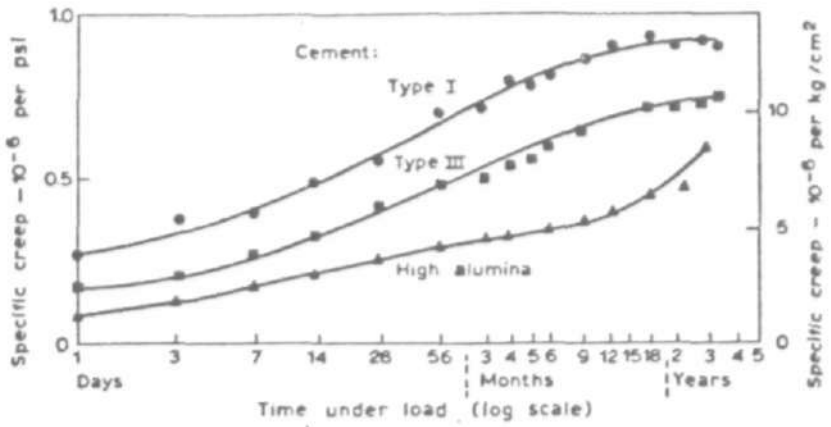


A 11 /116,319/



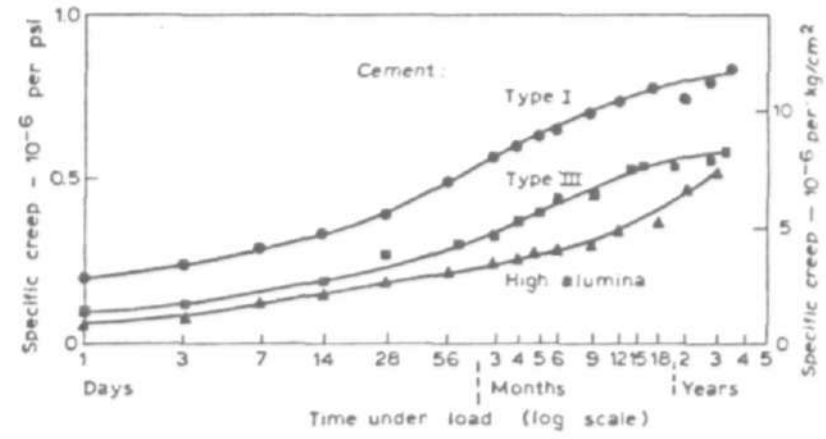
Static modulus of elasticity of concretes made with gravel aggregate and expanded clay aggregate, and tested at different ages up to one yea

A 1 2 /28/



Specific creep of concretes made with different cements, loaded at the age of 3 days

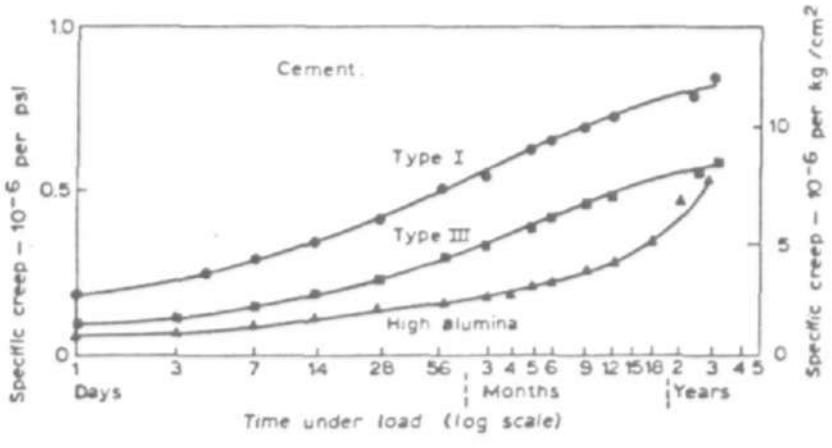
A 13 /201,286 /



Specific creep of concretes made with different cements, loaded at the age of 28 days

/201,286/.

A 14



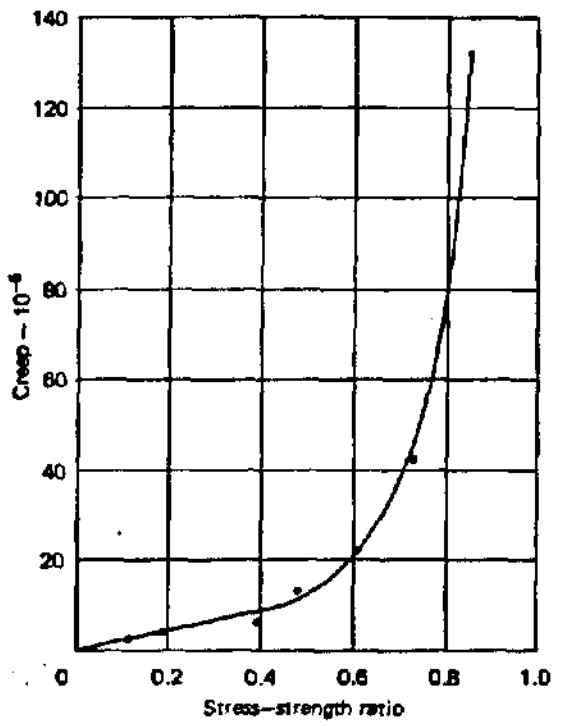
Specific creep of concretes made with different cements, loaded at the age of 90 days

A 15 /201,286/.

Details of mixes used in the comparison of creep of concretes made with different cements from Hummel's tests

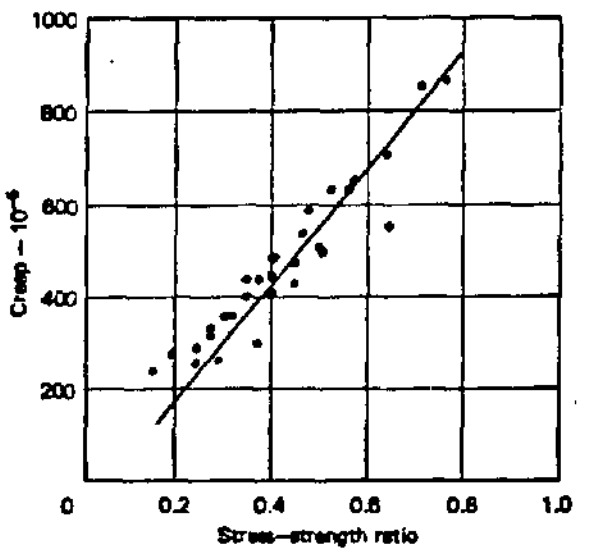
Age at loading (days)	Type of cement	Strength at application of load		Extrapolated ultimate creep	
		psi	kg/cm²	10^{-6} per psi	10^{-6} per kg/cm²
3	I	2080	146	1.04	14.8
	III	5060	356	0.86	12.3
	high-alumina	7600	534	–	–
28	I	4600	323	0.83	11.8
	III	7150	504	0.56	8.0
	high-alumina	9550	672	–	–
90	I	5780	407	0.83	11.8
	III	8150	574	0.56	8.0
	high-alumina	9900	695	–	–

A 16 /286/.



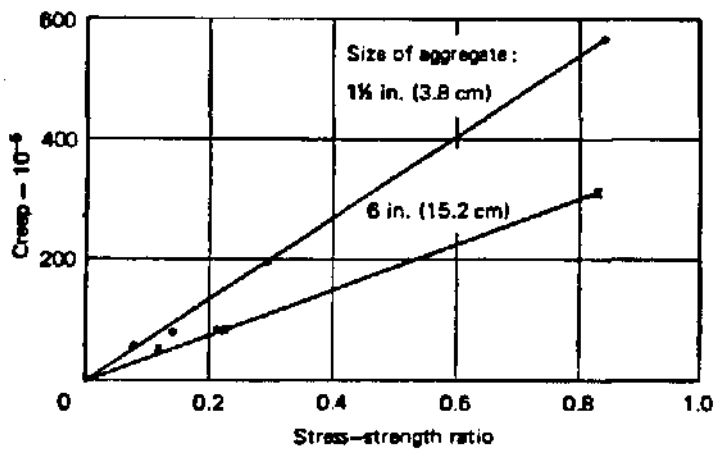
Relation between creep after 1 minute under load and stress–strength ratio for 1:4:5 concrete, loaded at the age of 7 days.

A 17 /286/.



Relation between creep of mortar stored moist after 60 days under load and the stress–strength ratio for different mixes and stresses.

A 18 /285/.



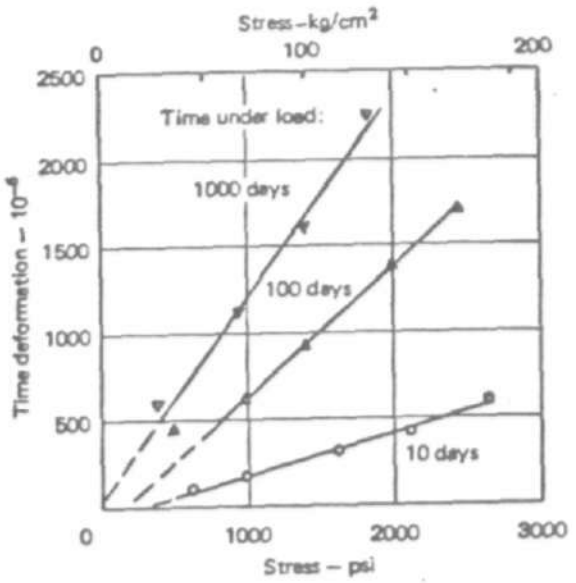
Relation between creep and stress–strength ratio derived from measurements of the modulus of elasticity (eq.

A 19 /286/.

Creep of concretes of different strengths

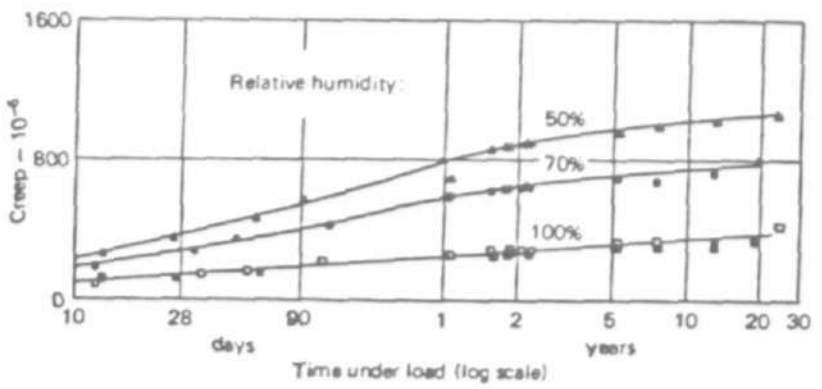
Compressive strength at time of application of load (psi)	Ultimate specific creep (10^{-6} per psi)	Ultimate creep at a stress–strength ratio of 30 per cent (10^{-6})
2000	1.40	933
4000	0.80	1067
6000	0.55	1100
8000	0.40	1067

A 20 /224/.



Relation between total time-deformation and stress for concrete stored at a relative humidity of 75 per cent .

A 21 /232/.



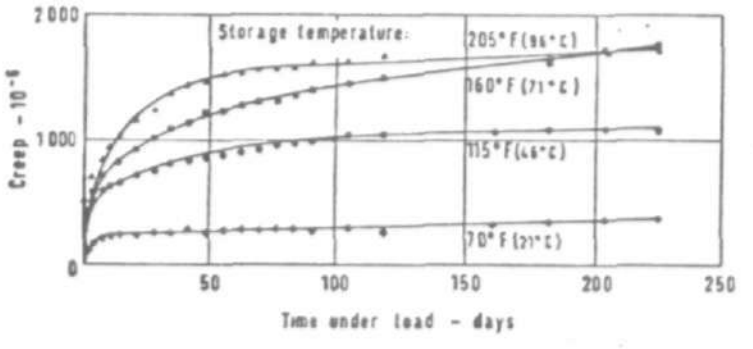
Influence of relative humidity of storage on creep of concrete (4 by 14 in. (10 by 35 cm) cylinders; 1 : 5.67 mix with a water-cement ratio of 0.59; moist-cured for 28 days and then loaded at the humidity indicated)

A 22 /377/.

Creep at different temperatures after 15 months under load

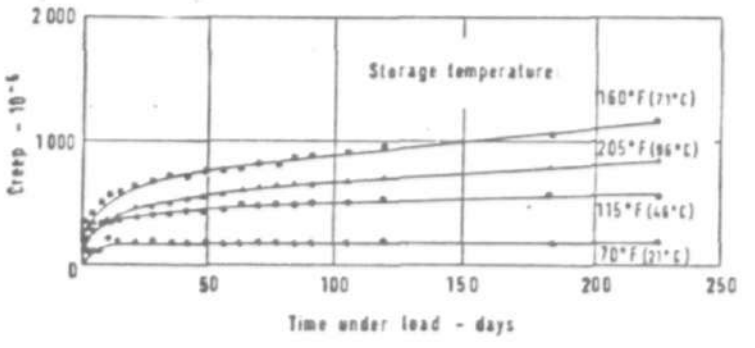
Stress–strength ratio	Creep at 70°F (21°C) (10^{-6})	Ratio of creep at temperature shown to creep at 70°F (21°C)		
		115°F (46°C)	160°F (71°C)	205°F (96°C)
0.35	630	1.32	1.75	1.95
0.60	1220	1.10	2.16	1.63
0.70	1560	1.07	2.23	1.42

A 23 /333,269/.



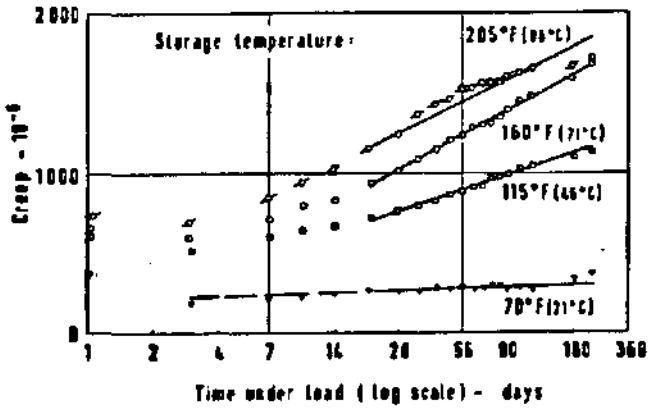
Relation between creep and time under load for concrete heated one week before loading at the age of one year.

A 24 /269/.



Relation between creep and time under load for concrete heated 14 days before loading at the age of 50 years.

A 25 /269/.



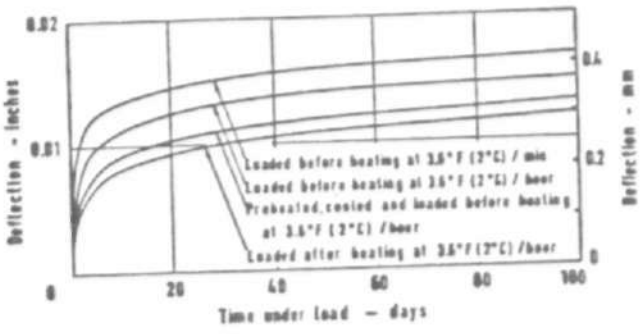
Relation between creep and logarithm of time under load for concrete heated one week before loading at the age of one year to a stress–strength ratio of 0.45.

A 26 /269/.

Creep after 225 days under load at different temperatures

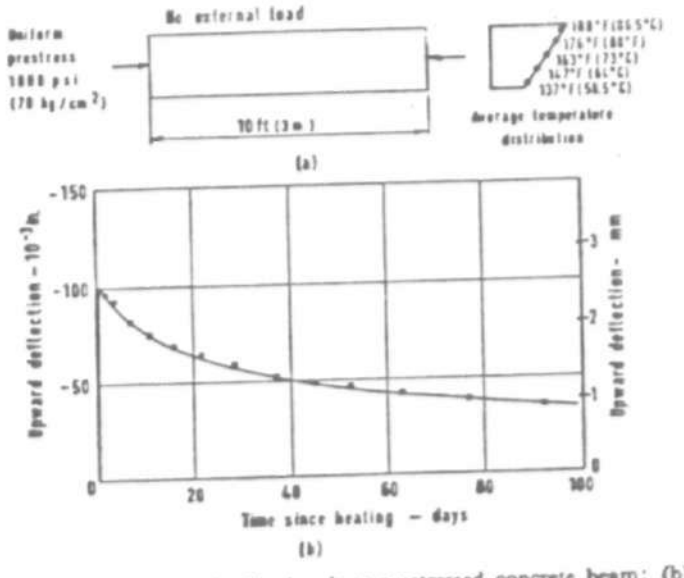
Age at loading (years)	Storage temperature before application of load °F (°C))	Stress–strength ratio (per cent)	Creep at 70°F (21°C) (10^{-6})	Ratio of creep at temperature shown to creep at 70°F		
				115°F (46°C)	160°F (71°C)	205°F (96°C)
1	as under load	25	190	1.74	2.53	0.79
		45	350	1.91	2.28	1.54
1	70 (21)	25	190	3.42	3.26	5.10
		45	350	3.34	4.92	4.83
50	outdoors	45	210	2.72	5.47	4.04

A 27 /269,286/.



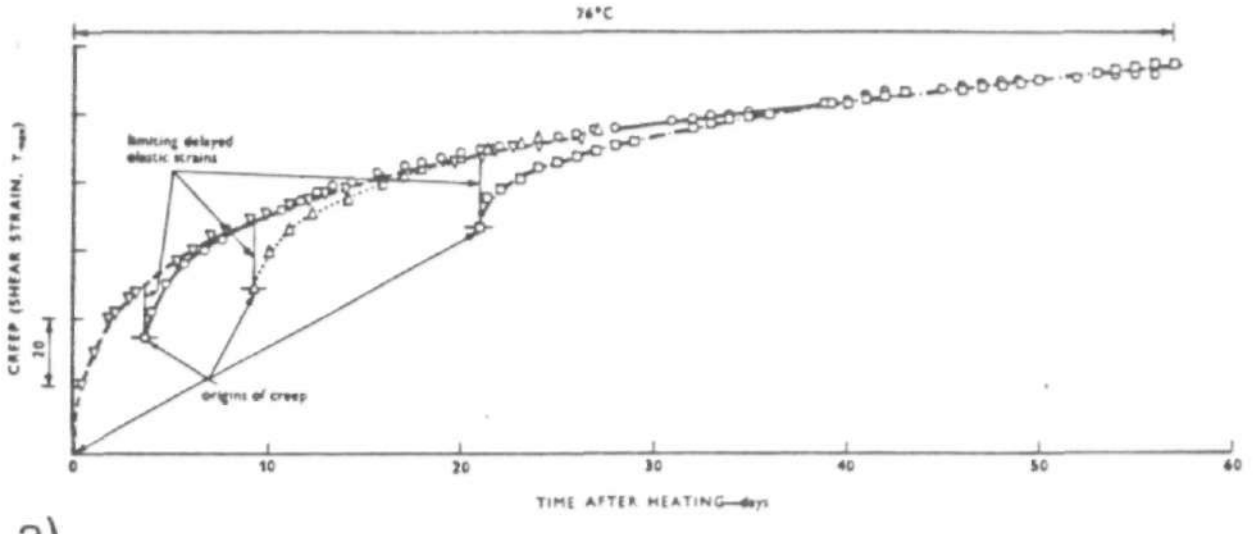
Influence on creep of the sequence of heating and loading

A 28 /181/.



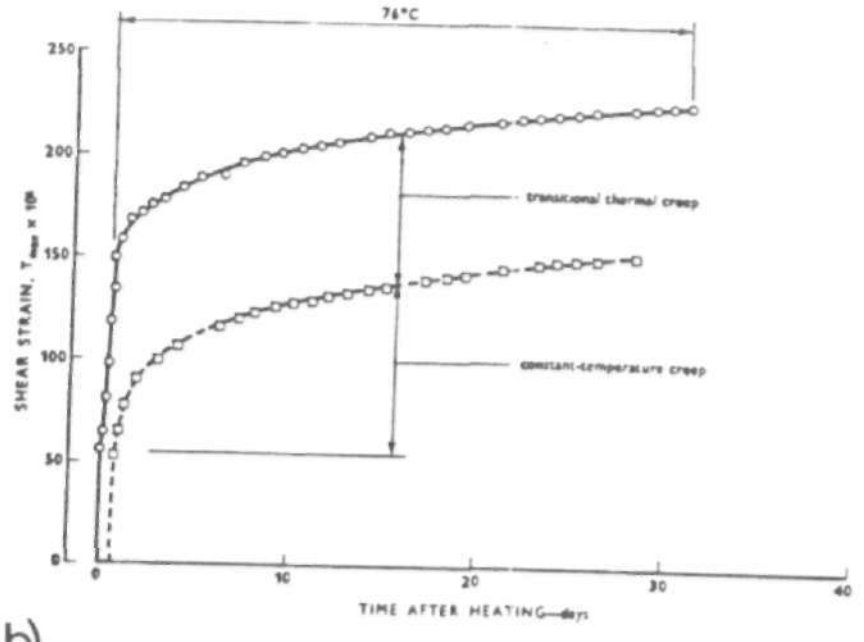
(a) Temperature distribution in a prestressed concrete beam; (b) effect of heating on deflection

A 29 /334/.

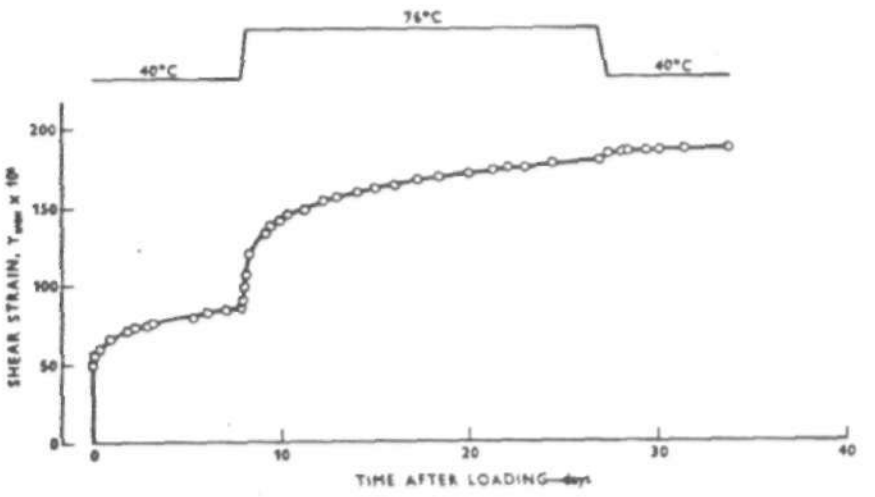


a) *Constant-temperature creep curves. Specimens heated to 76°C at 40 days and loaded 18 h, 3·6 days, 9·3 days and 21 days later.*

/205/.

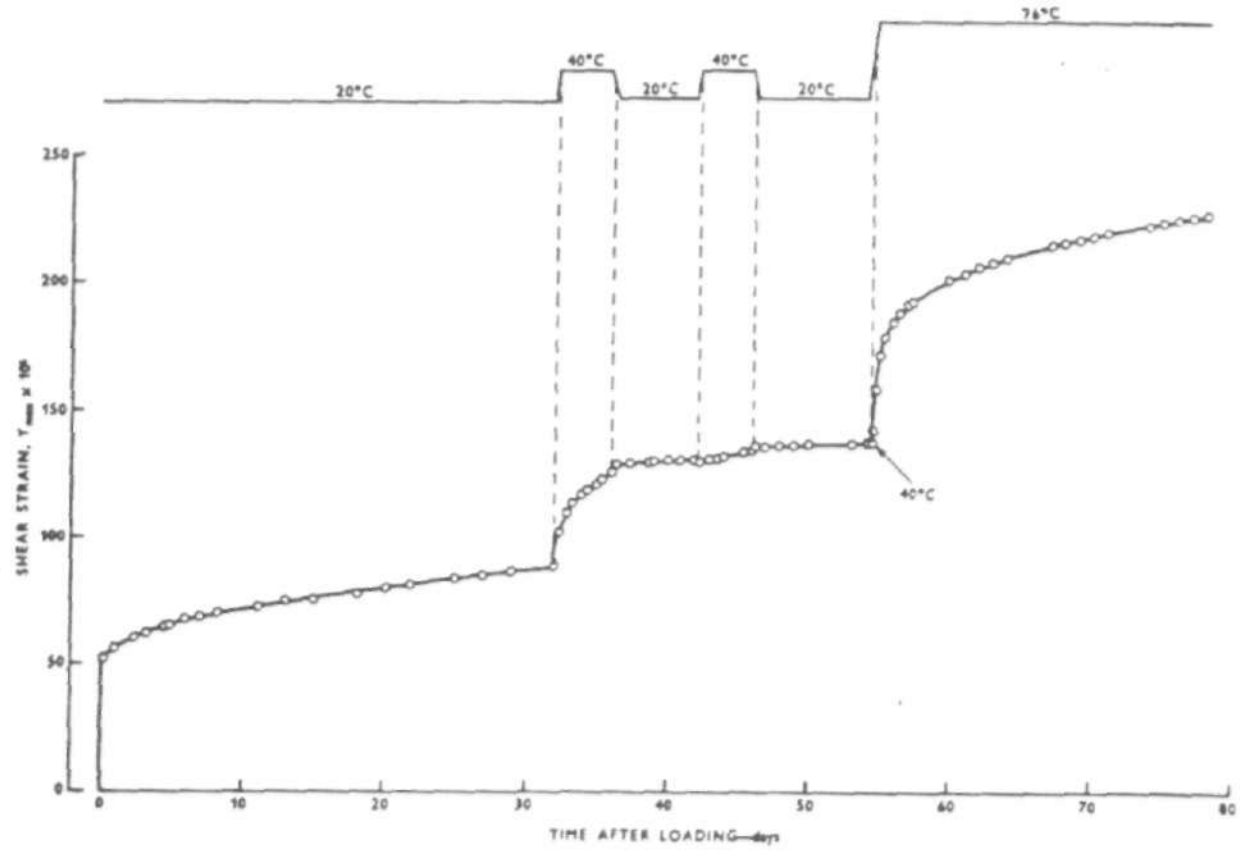


b) *Strains of mortar specimens loaded just before and just after heating. Age at heating: 40 days.*



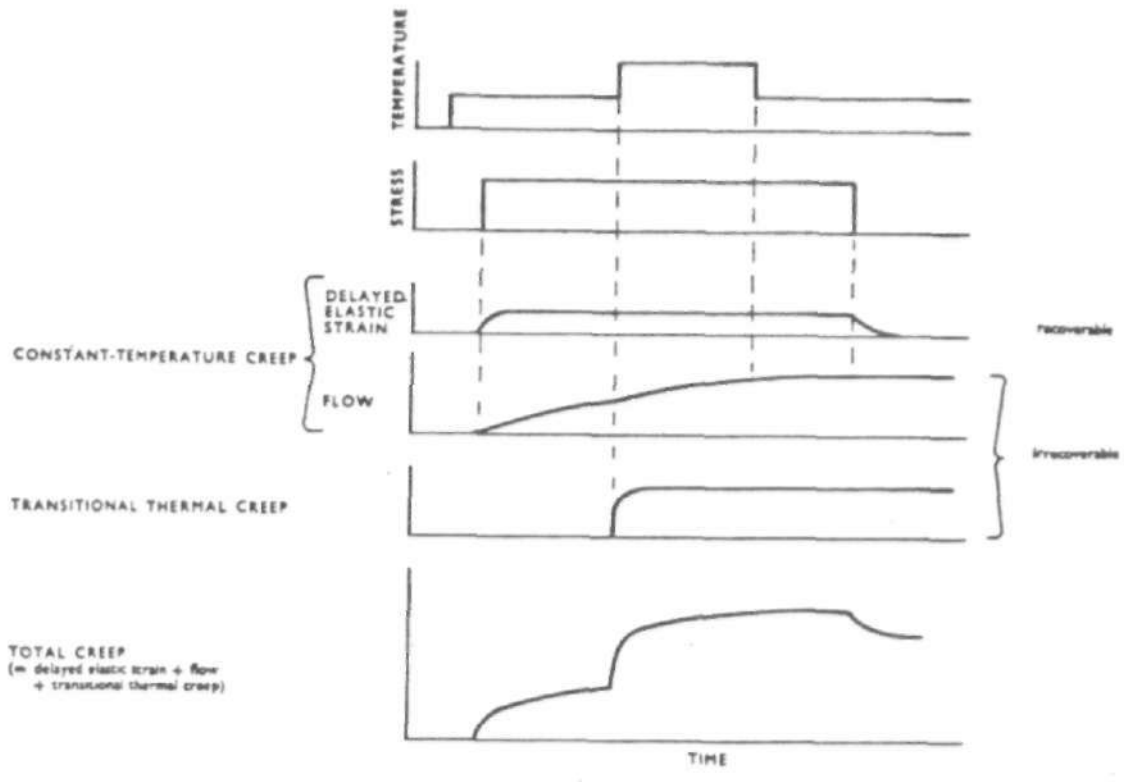
The effect upon creep of a decrease in temperature.

A 31 /205/.



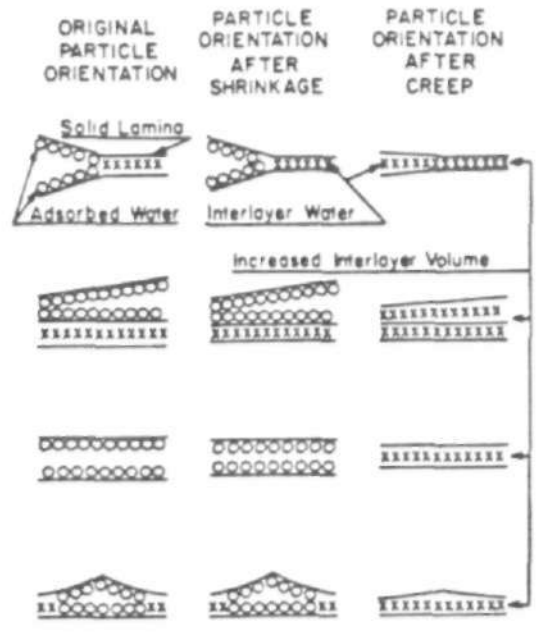
The effect upon creep of cycles of temperature.

A 32 /205/.



Diagrammatic summary of creep components under varying temperature.

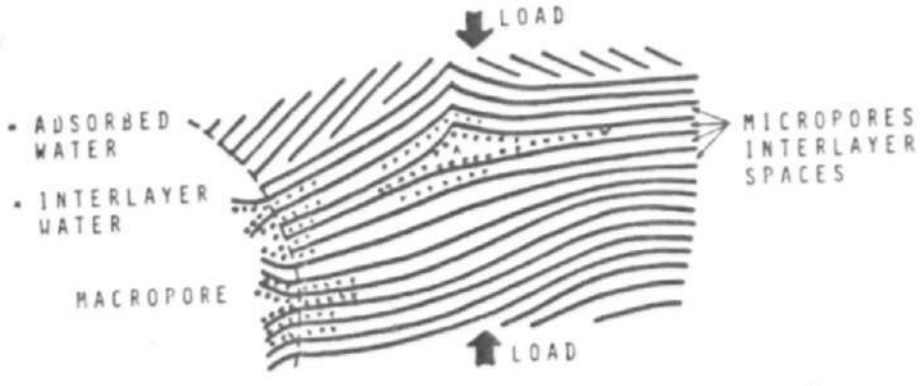
A 33 /205/.



MODEL FOR CREEP OF CEMENT PASTE

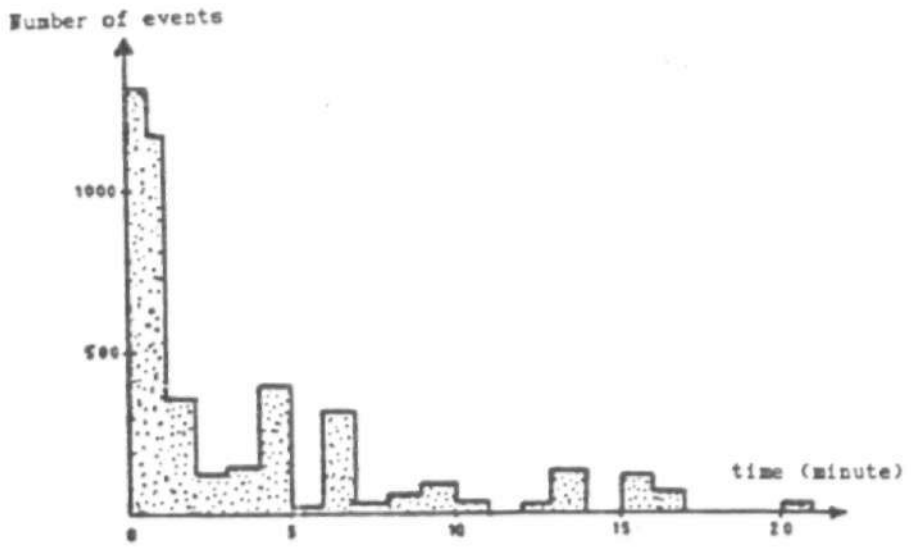
A 34

/198/.



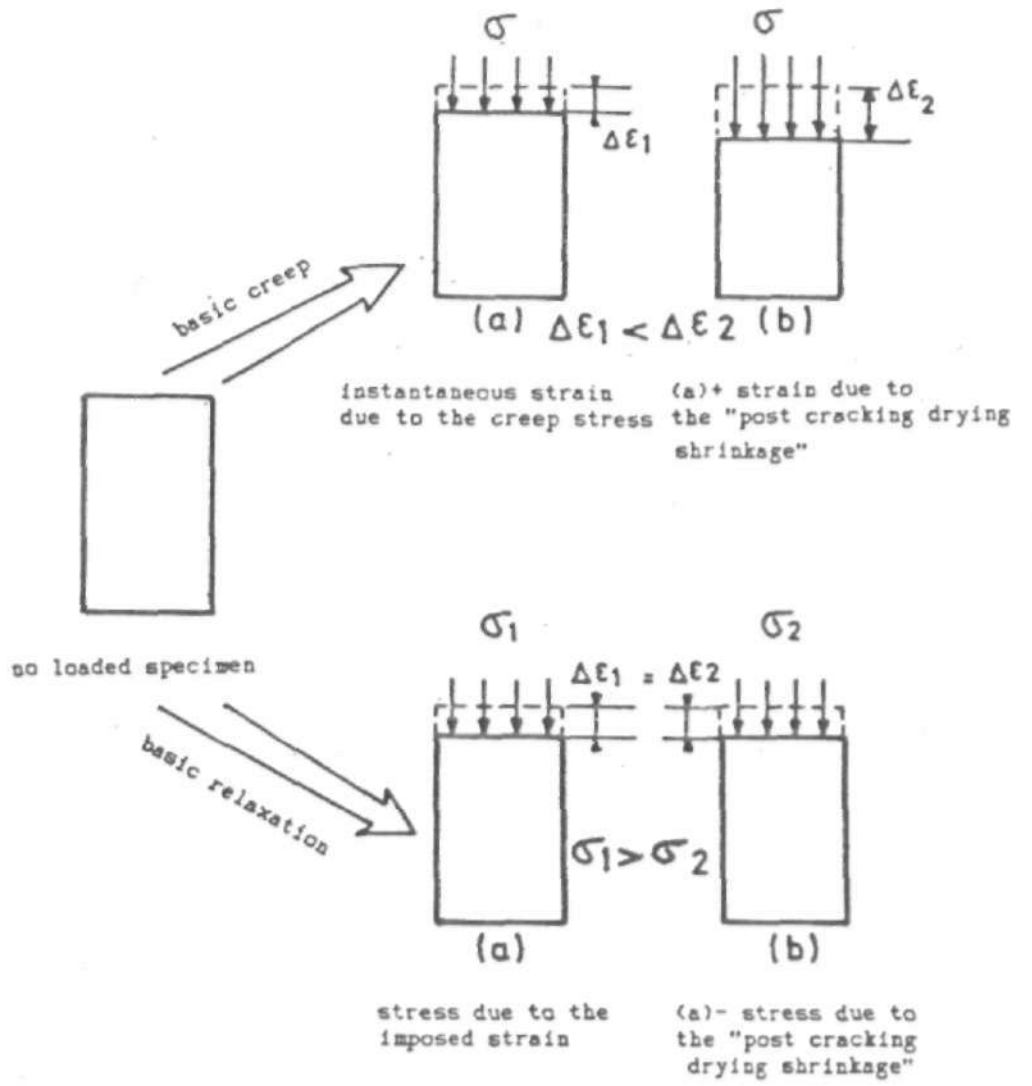
Idealized Typical Arrangement of a Number of Sheets of Solids (Silicates) in Cement Paste

A 35 /37/.

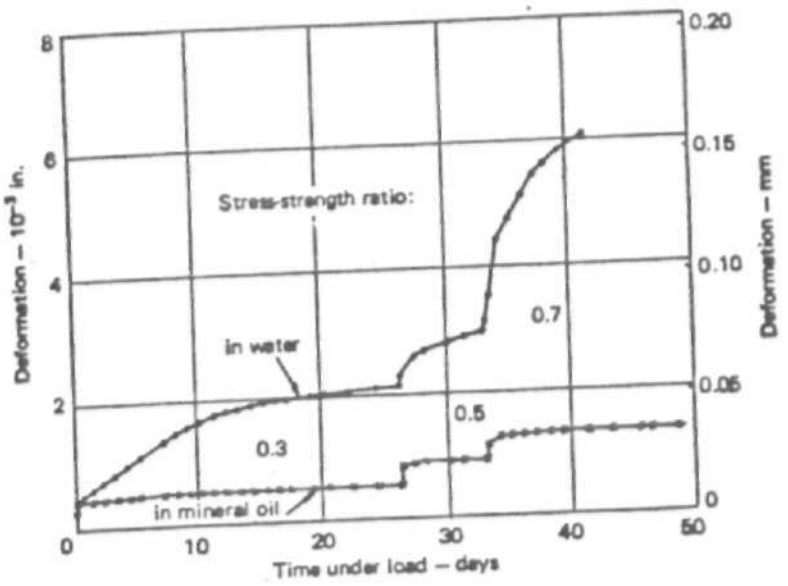


Acoustic emission obtained during a basic creep test.

A 36 /335/.

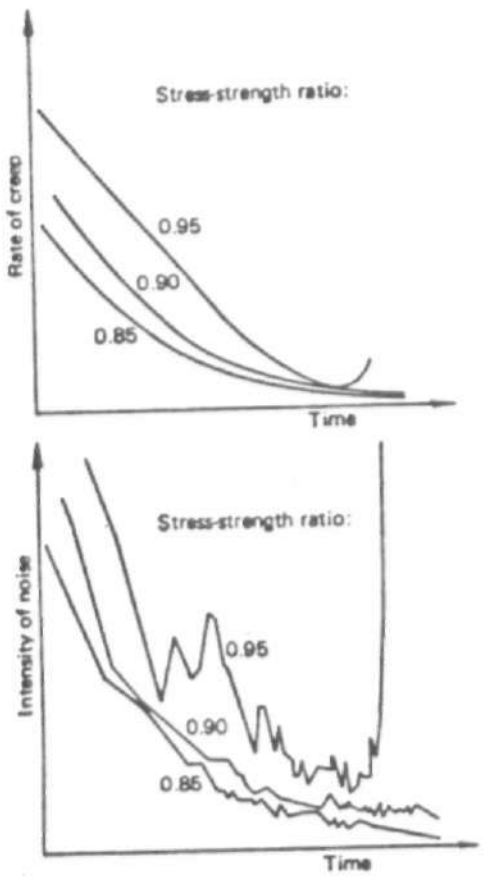


Main features of basic creep and relaxation based on the hypothesis of the "post cracking drying shrinkage"



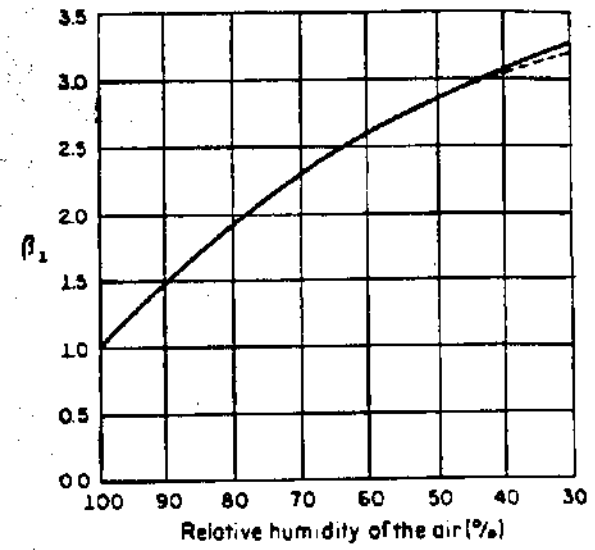
Creep of mortar beams in mineral oil and in water

A 38 /286/.

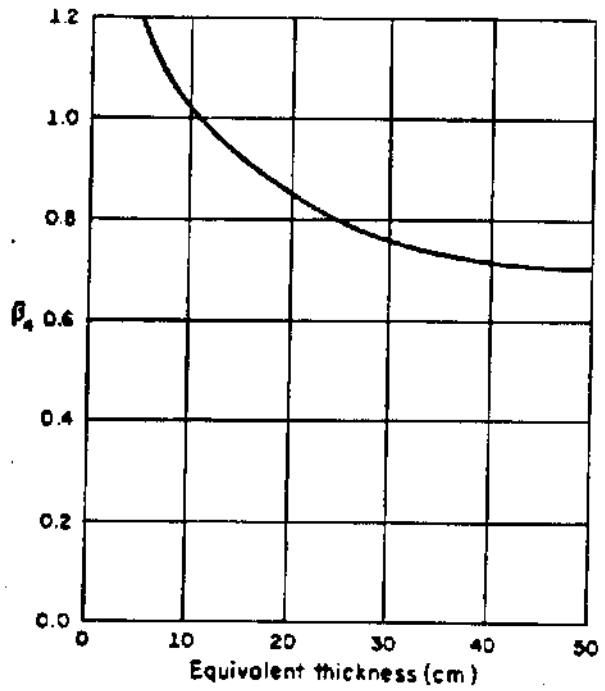


Affinity between the rate of creep and the intensity of internal noise during the first day of loading

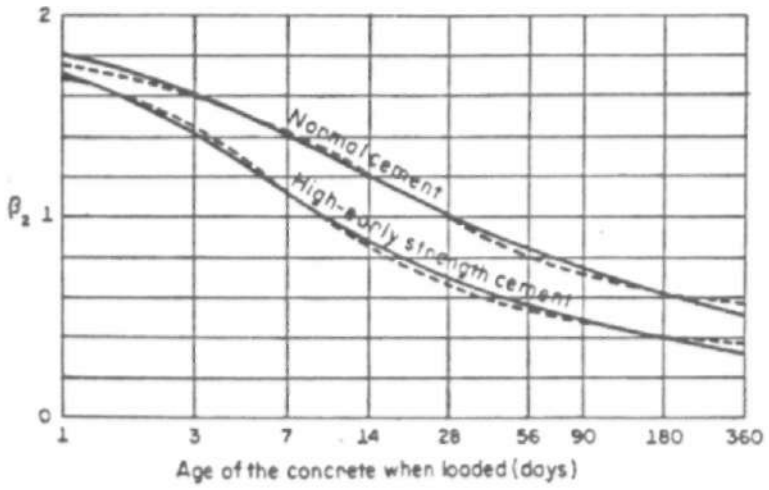
A 39 /340/.



a) – Coefficient β_1. CEB-FIP Recommendations. /300/.

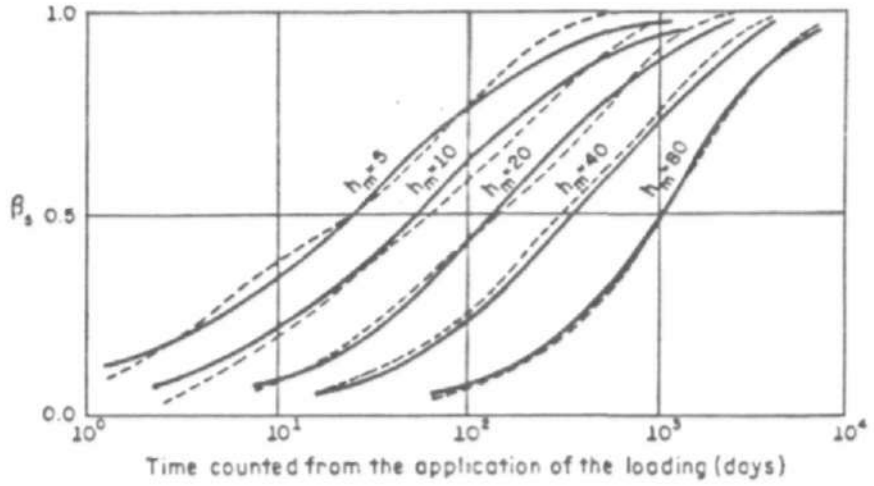


b) – Coefficient β_4. CEB-FIP Recommendations.



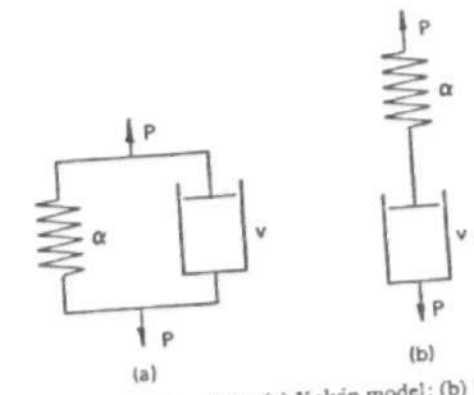
c) – Coefficient β_2. CEB-FIP Recommendations.

/300/.



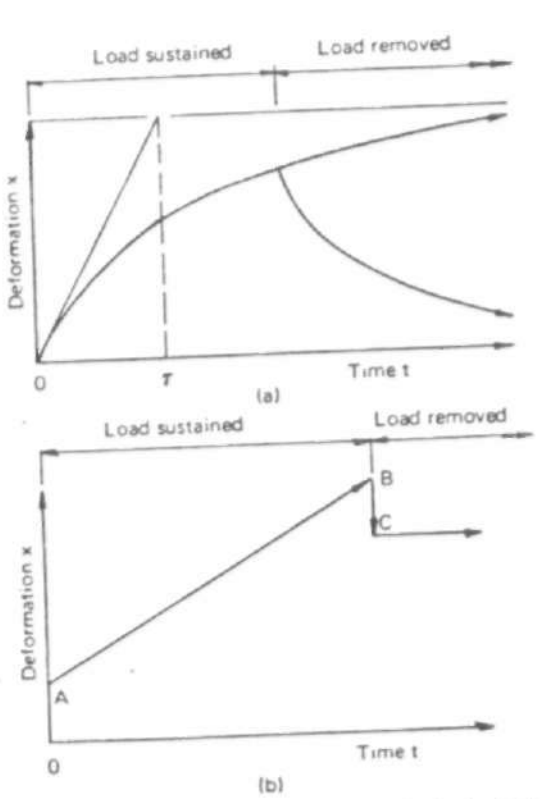
d) – Coefficient β_3. CEB-FIP Recommendations.

A 40 cont!d. /300/.



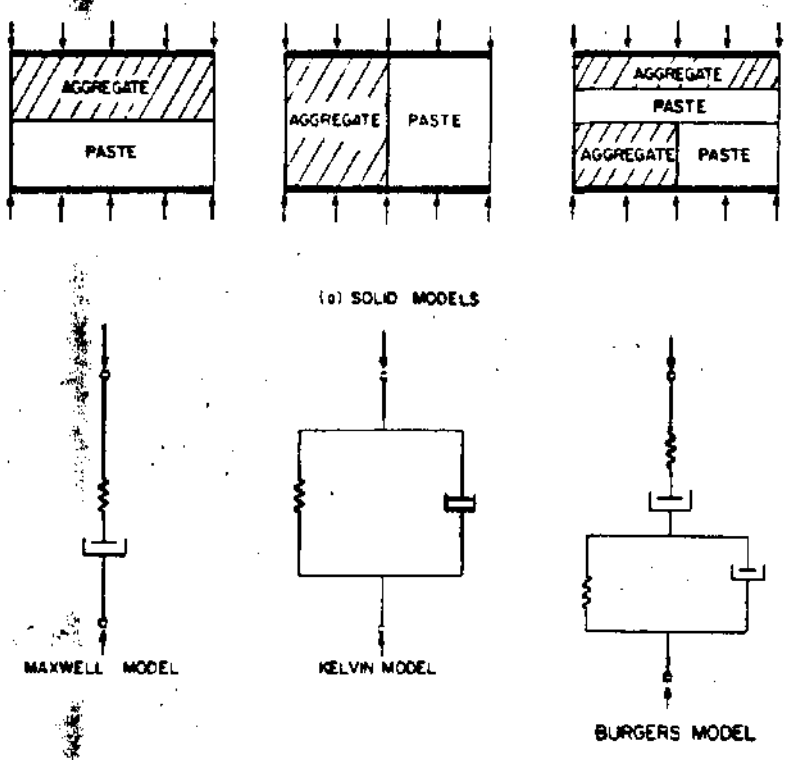
Basic rheological models: (a) Kelvin model; (b) Maxwell model.

A 41 /286/.

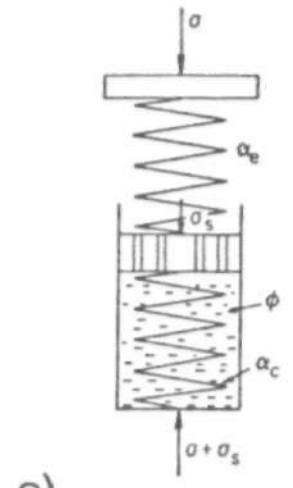


The deformational response of basic rheological models:
(a) Kelvin model; (b) Maxwell model.

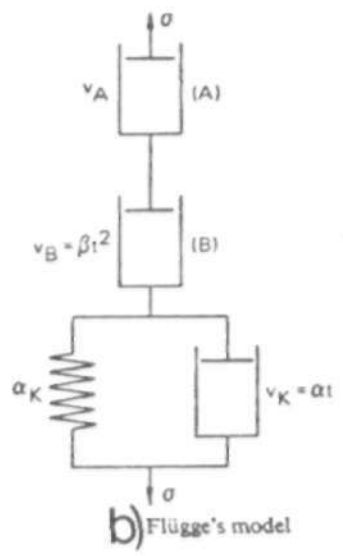
A 42 /286/.



Solid models and their viscoelastic counterparts.

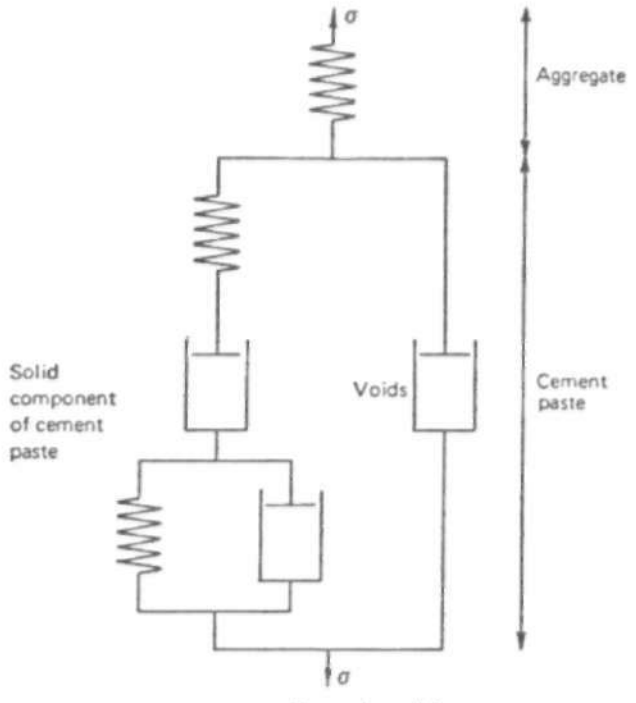


a) Ross's model /327/.



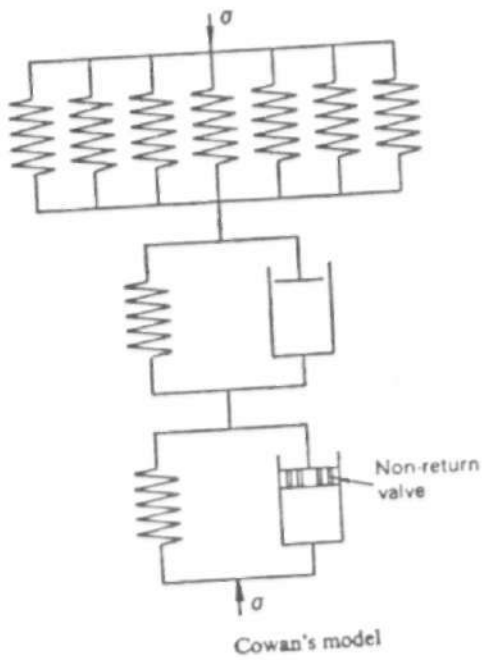
b) Flügge's model /136/.

A 46 /136,327/.



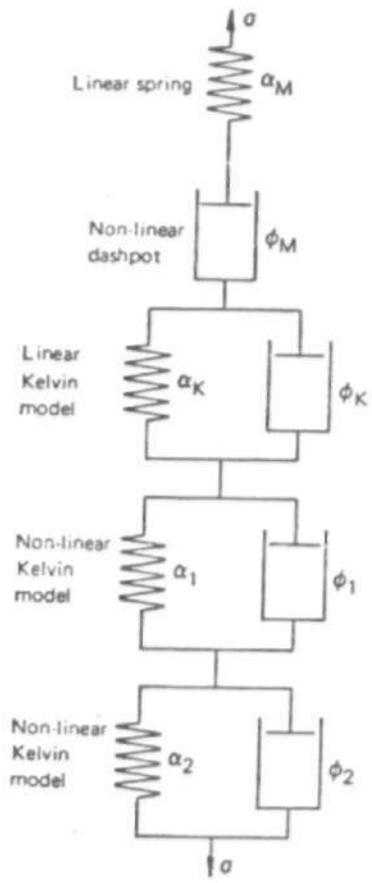
Hansen's model

A 47 /175/.

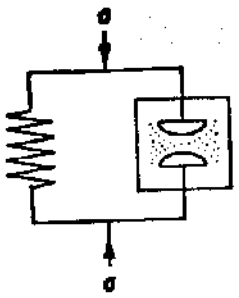


Cowan's model

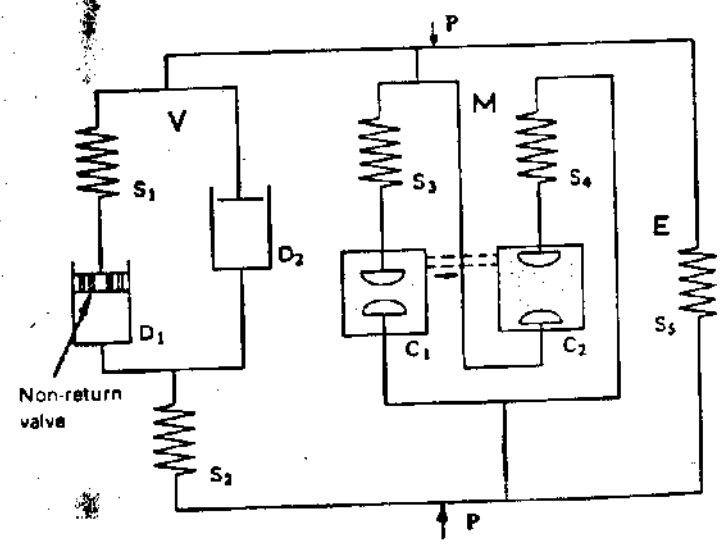
A 48 /94/.



Freudenthal ane Roll's model

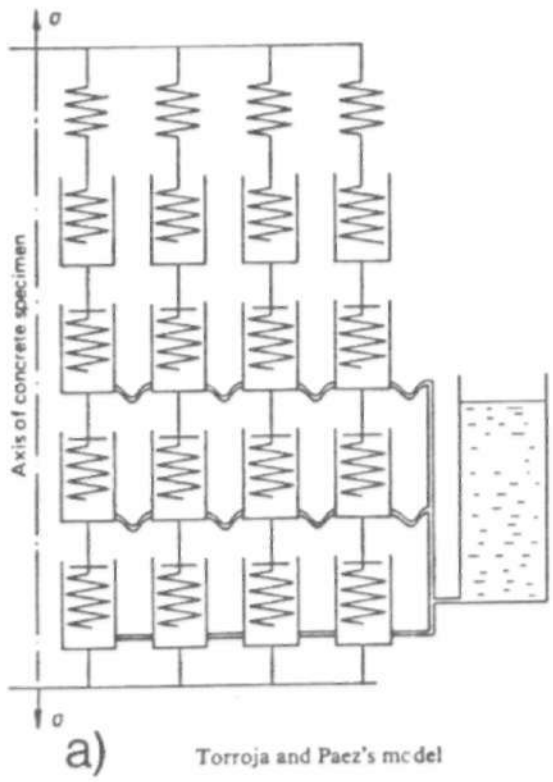


a) Powers' model /312/.

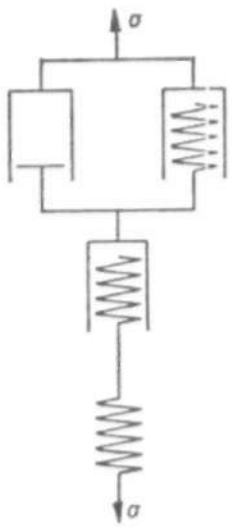


b) The Gopalakrishnan-Neville-Ghali model /165/.

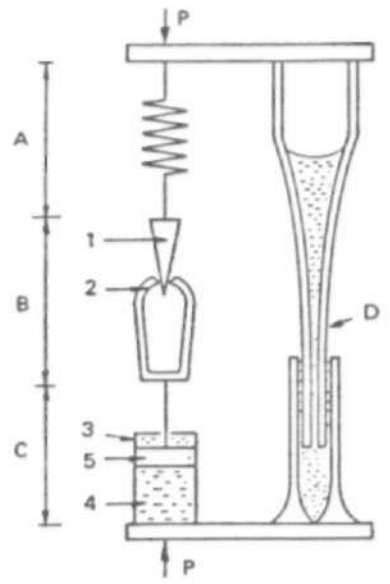
A 5 0 /165,312/.



a) Torroja and Paez's model /374/.

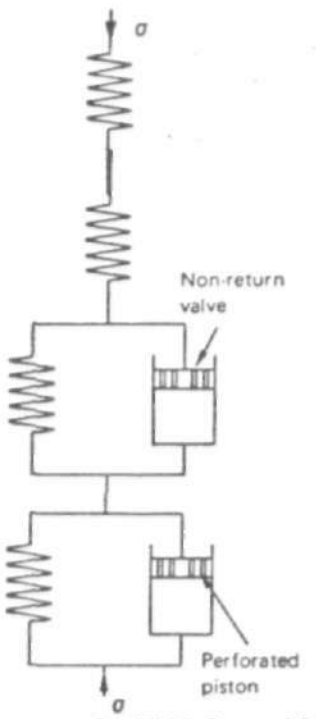


b) Elementary model of Torroja and Paez /374/.



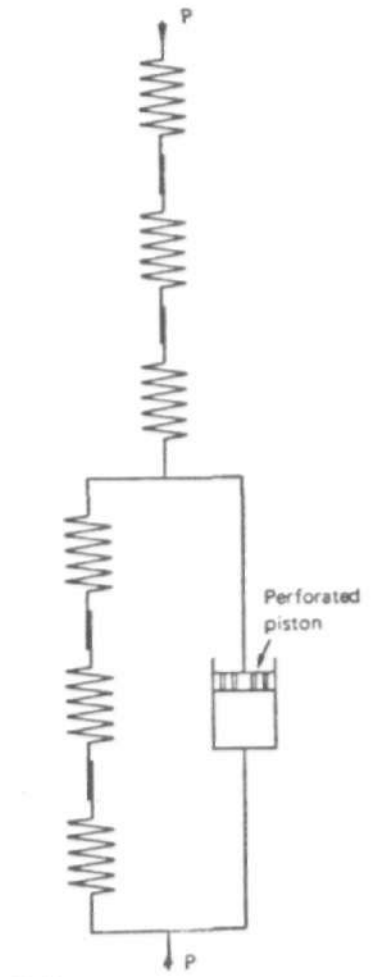
Bjuggren's model (A) elastic spring; (B) elastic deformation: 1 wedge, 2 elastic claw; (C) creep: 3 closed cylinder, 4 fluid, 5 porous piston; (D) telescopic capillary tubes.

A 52 /65/.

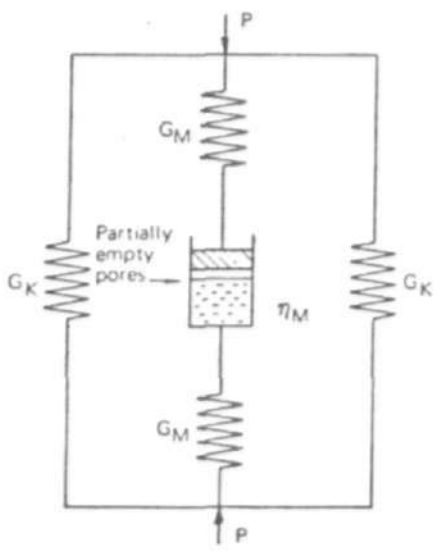


Glucklich's first model

A 53 /161/.

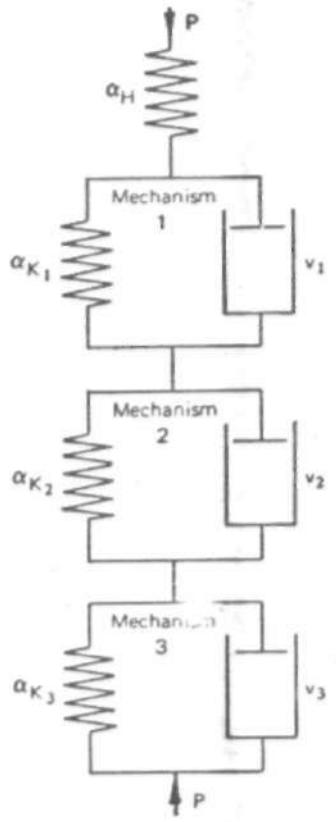


a) Glucklich's second model /161/.

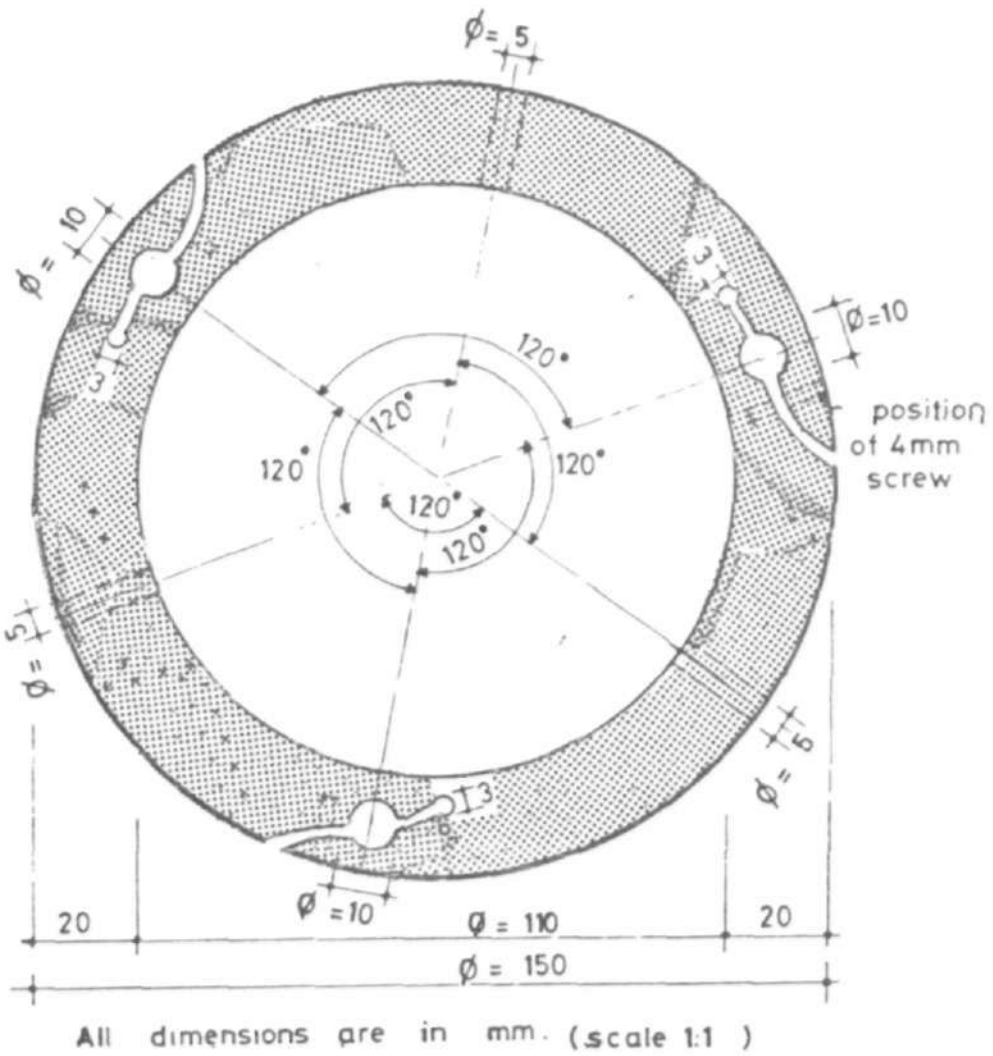


b) Glucklich and Ishai's model for creep in torsion /162/.

A 54 /161, 162/.



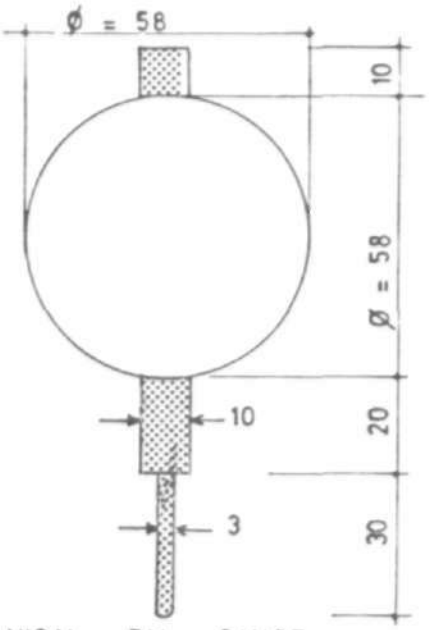
Ishai's model for creep in torsion



All dimensions are in mm. (scale 1:1)

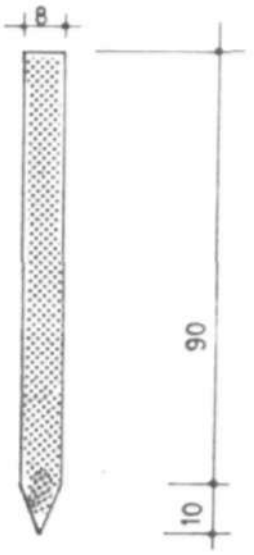
METAL RING FOR SPECIMEN PREPARATION

A 56



(a) MECHANICAL DIA GAUGE

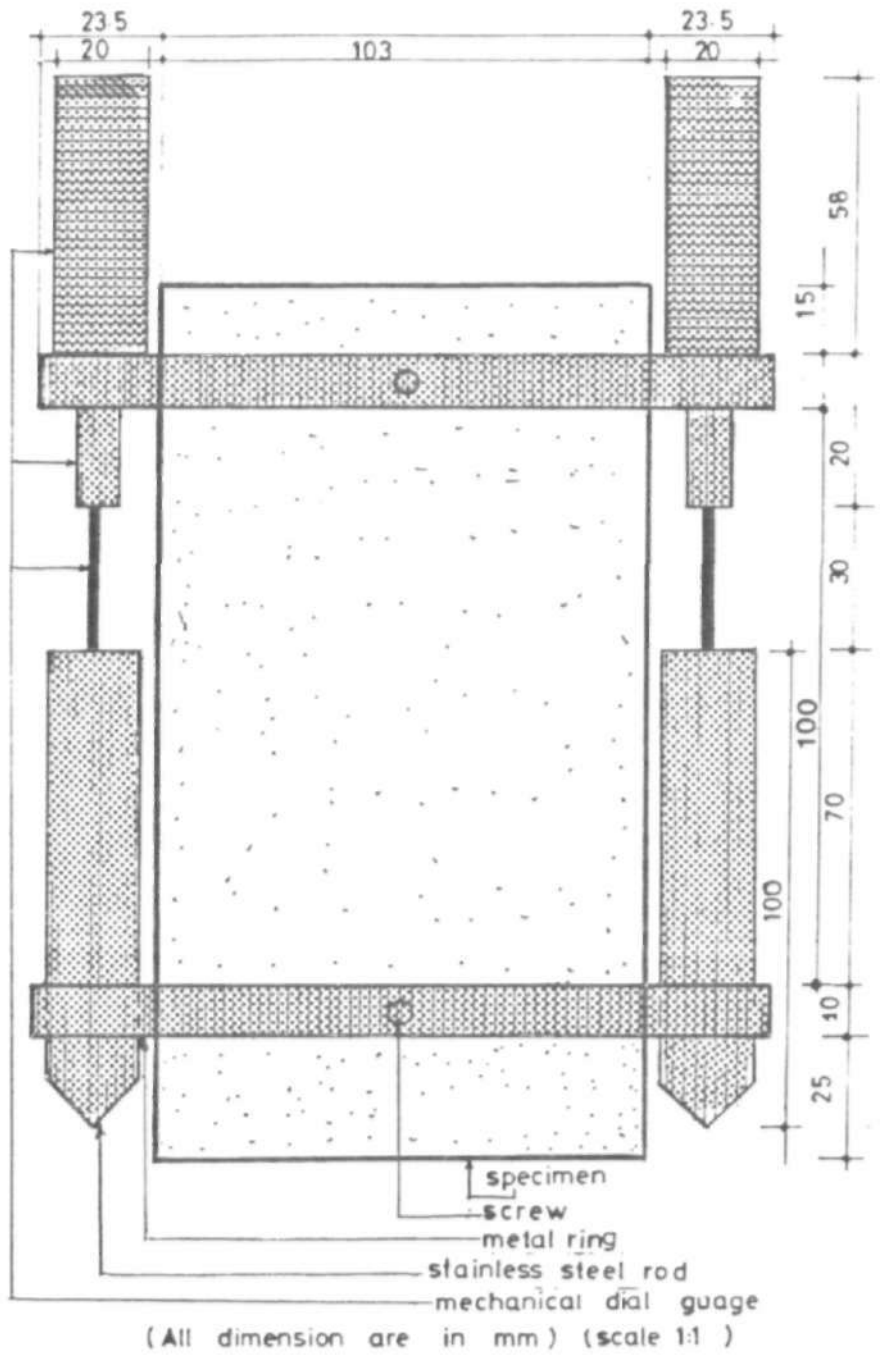
Scale 1:1



(b) Stainless Steel Rod (All dimensions in mm)

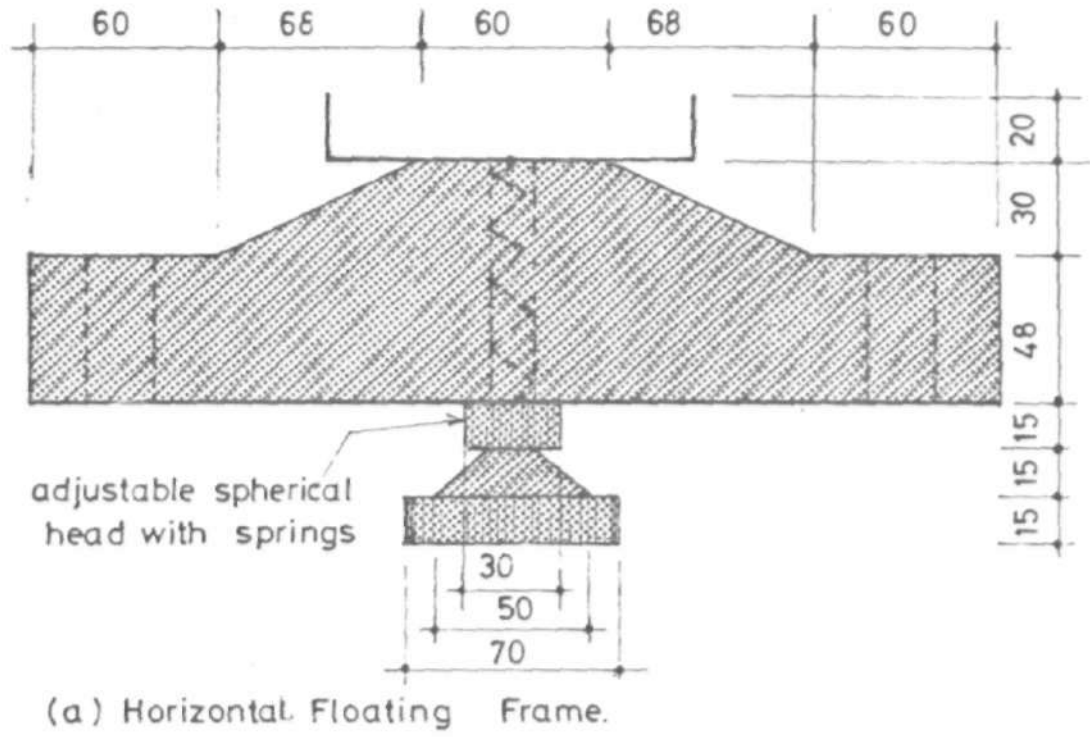
GAUGE AND STAINLESS ROD ON WHICH THE GAUGES SIT.

A 57

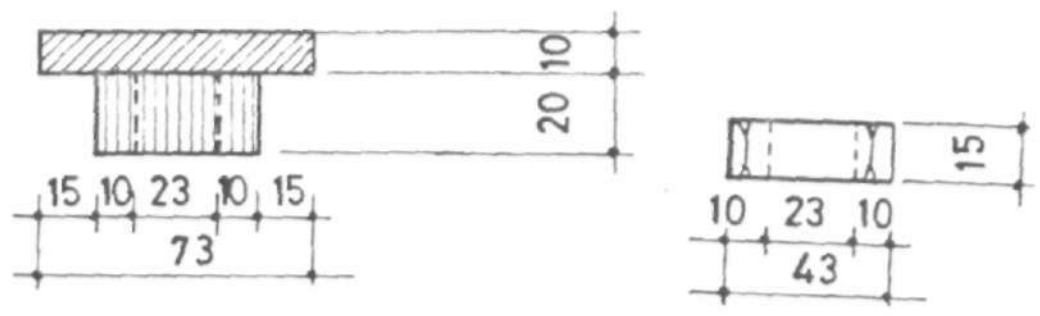


PREPARED SPECIMEN

A 58



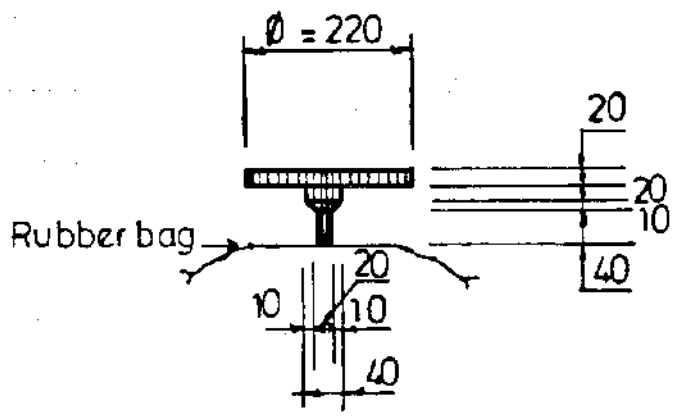
(a) Horizontal Floating Frame.



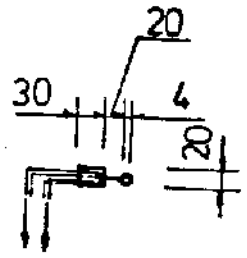
(b) Nuts. All dimensions are in mm) (scale 1:20)

HORIZONTAL FRAME SHOWING THE NUTS

A 59



(A) Loading Ram Connected to Rubber bag

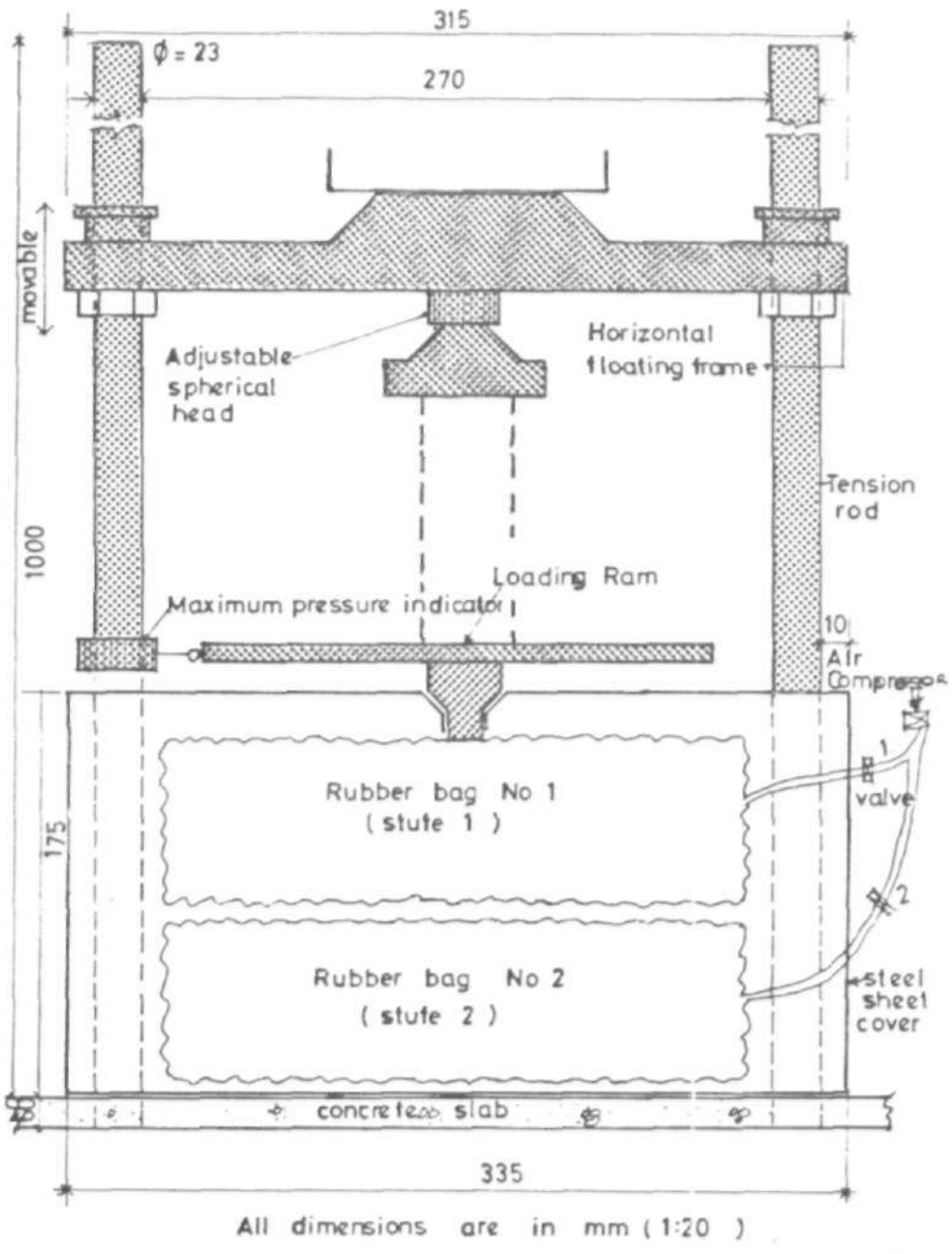


Two plastic hoses connected to each of the rubber bag 1 & 2

(B) Maximum pressure Indicator

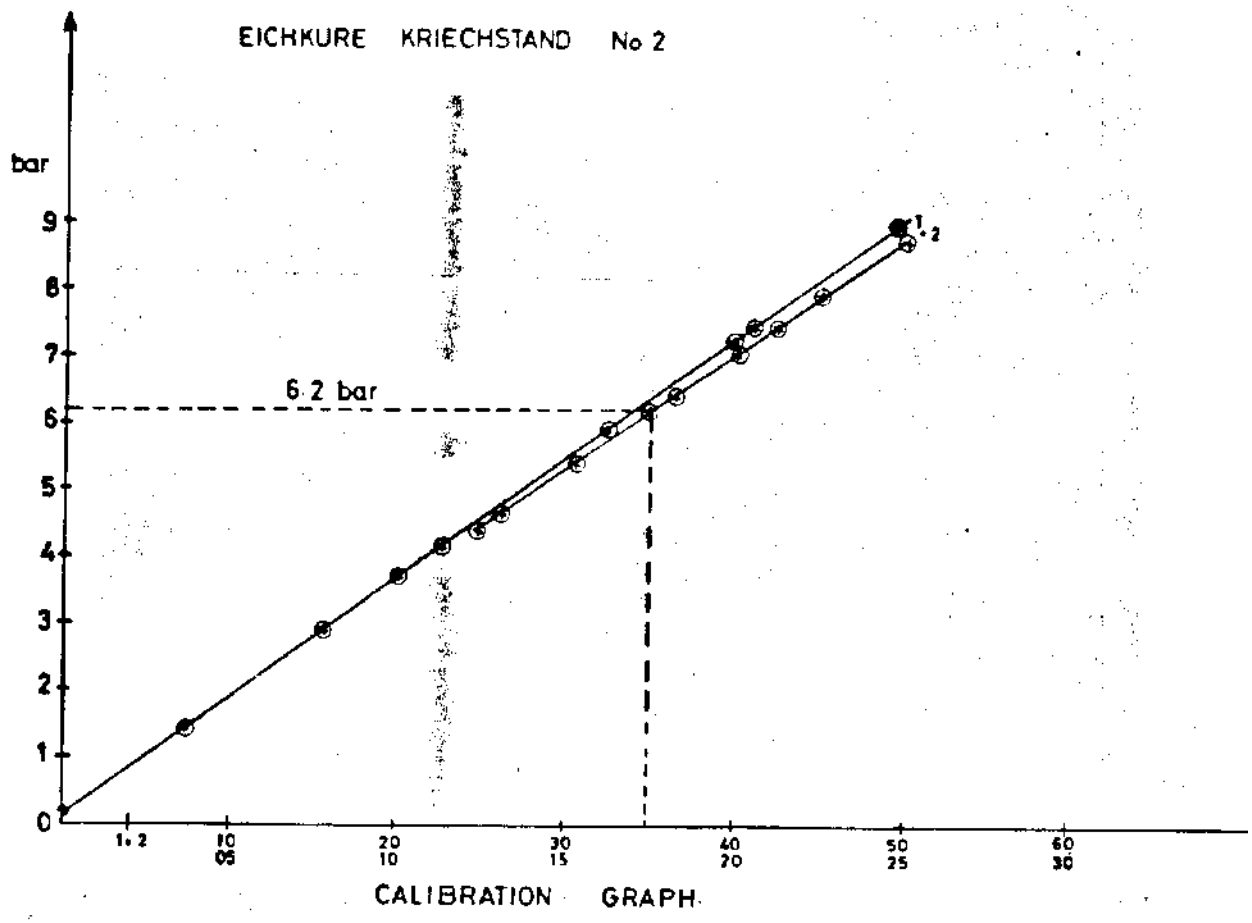
LOADING RAM AND THE MAXIMUM PRESSURE INDICATOR

A 60

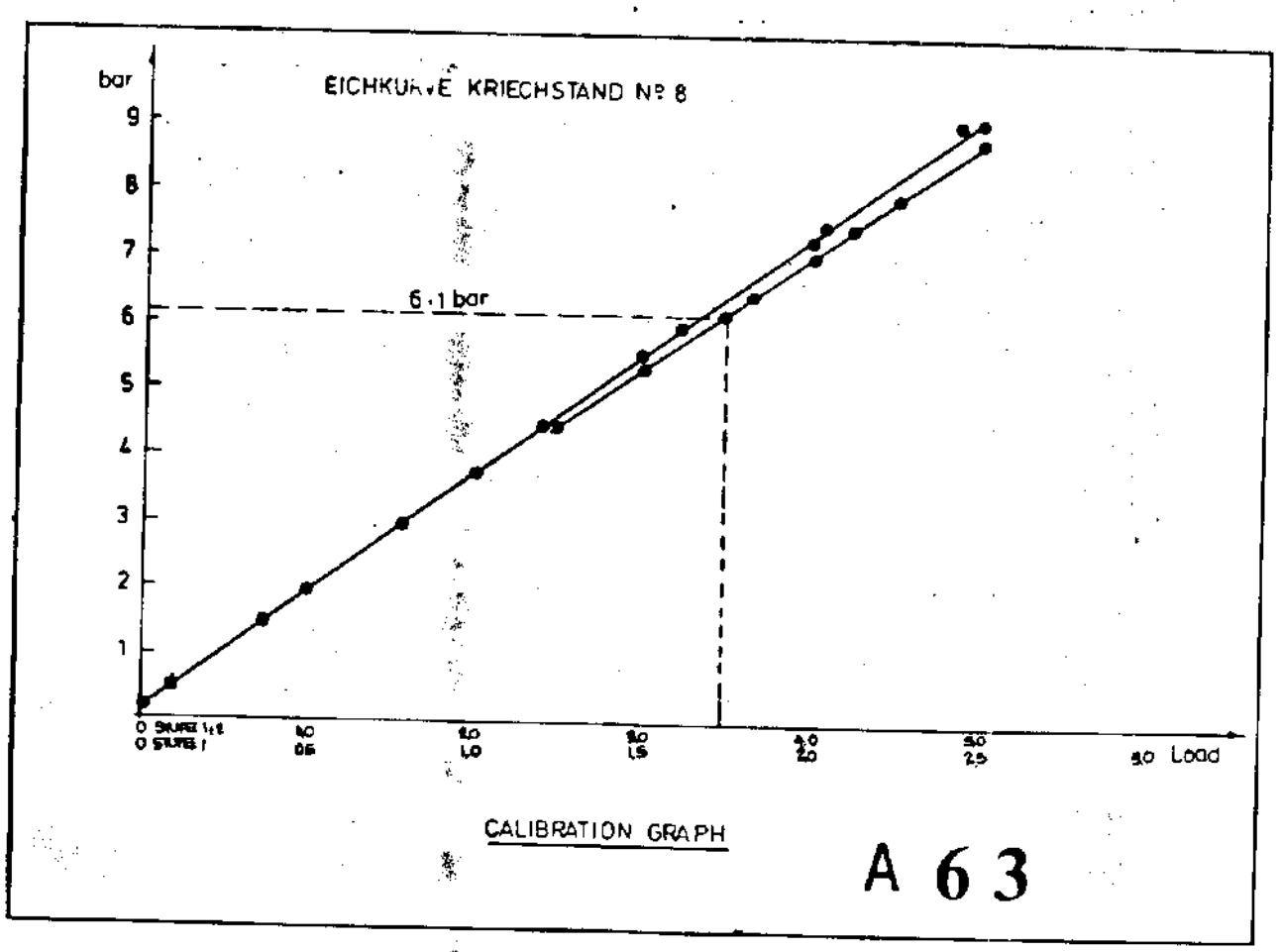


ASSEMBLED AIR HYDRADLIC CREEP FRAME

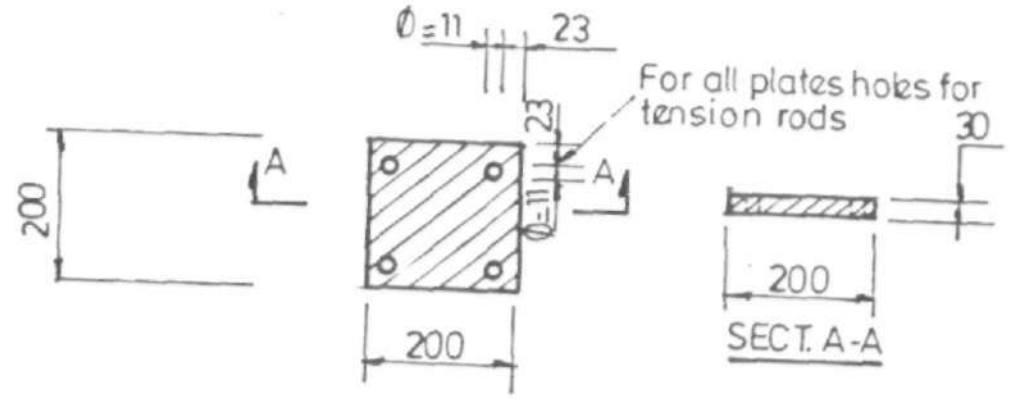
A 61


EICHKURE KRIECHSTAND No 2
bar
9
8
7
6.2 bar
6
5
4
3
2
1
0
1.2
10
05
20
10
30
15
40
20
50
25
60
30
CALIBRATION GRAPH

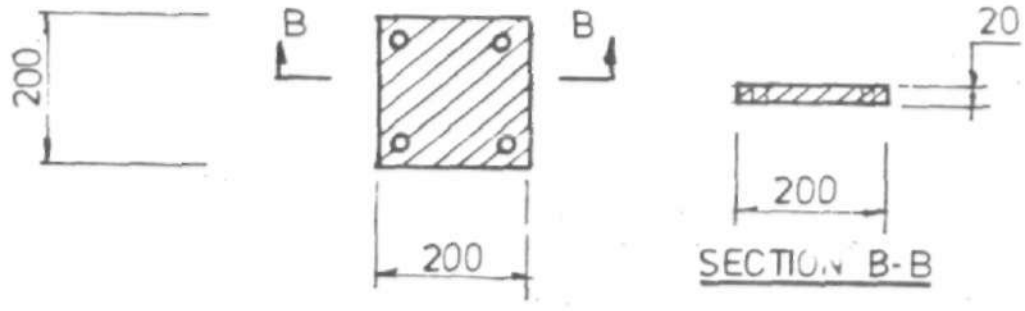
A 62


bar
EICHKURVE KRIECHSTAND Nº 8
9
8
7
6·1 bar
6
5
4
3
2
1
0 SKIRT 1/2
0 SKIRT 1
1.0
0.5
2.0
1.0
3.0
1.5
4.0
2.0
5.0
2.5
5.0 Load
CALIBRATION GRAPH

A 63



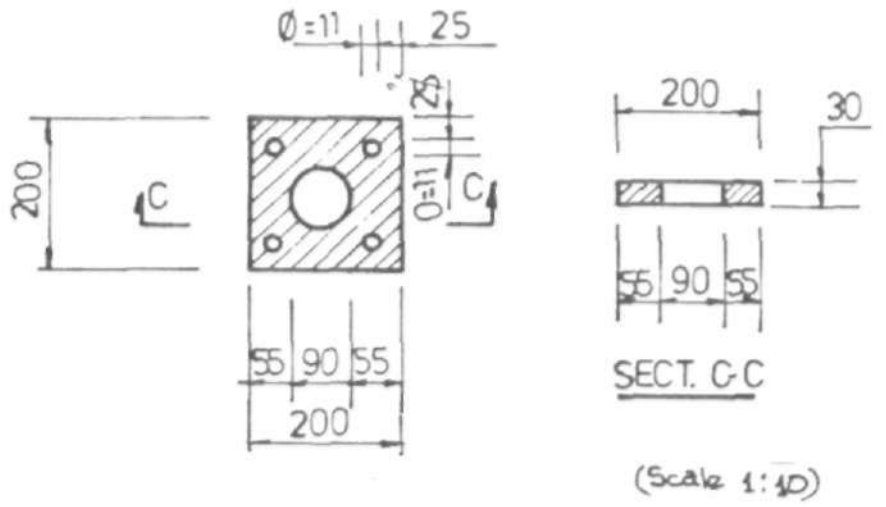
(A) Bottom plate



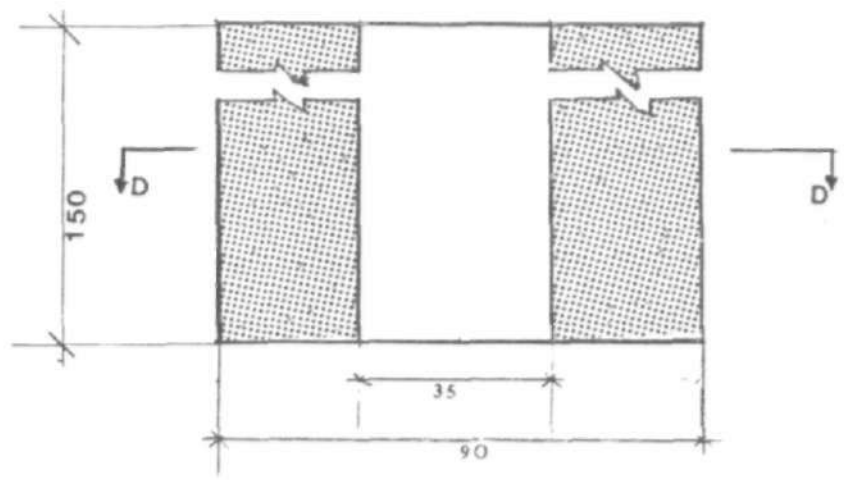
(B) Loading Ram plate

BOTTOM AND LOADING RAM PLATES
(Scale 1:10)

A 64

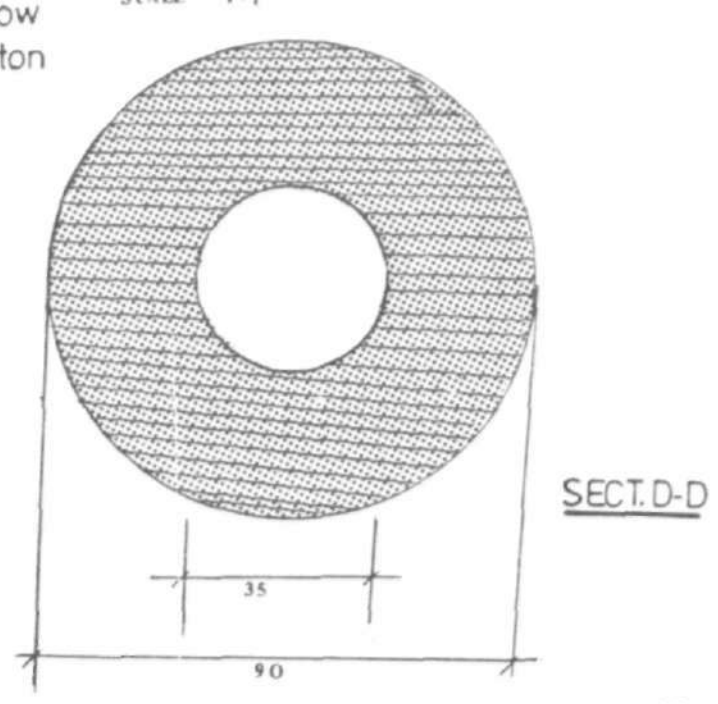


(A) Top Loading Plate



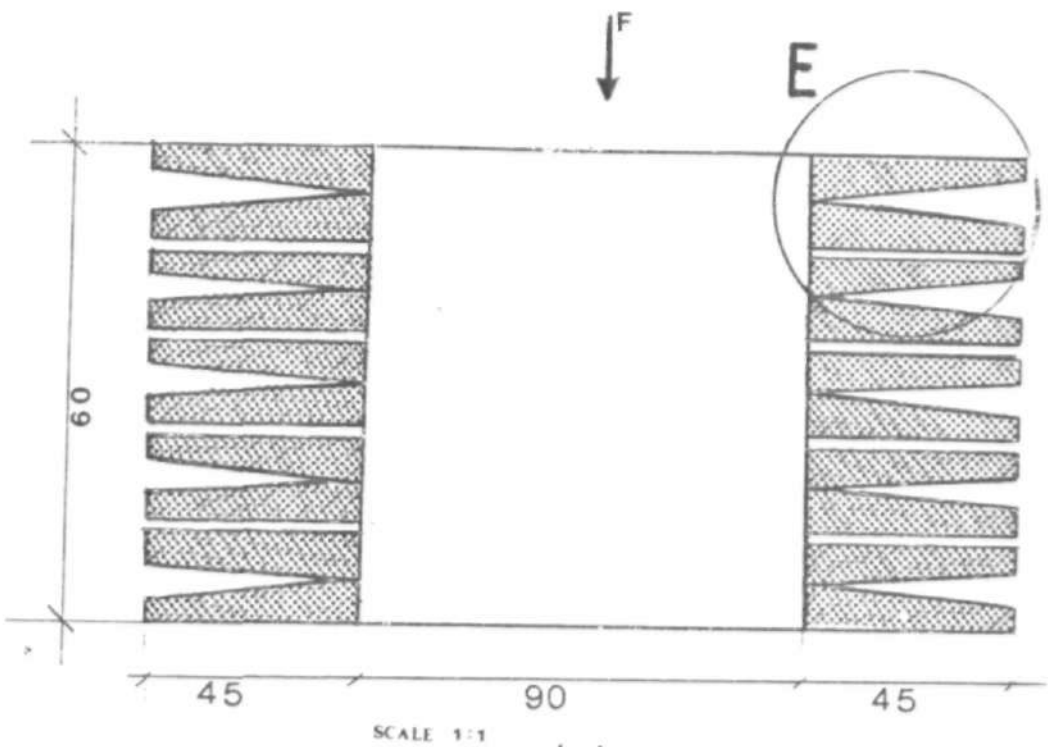
(B) Stainless Hollow Cylindrical Piston

SCALE 1:1



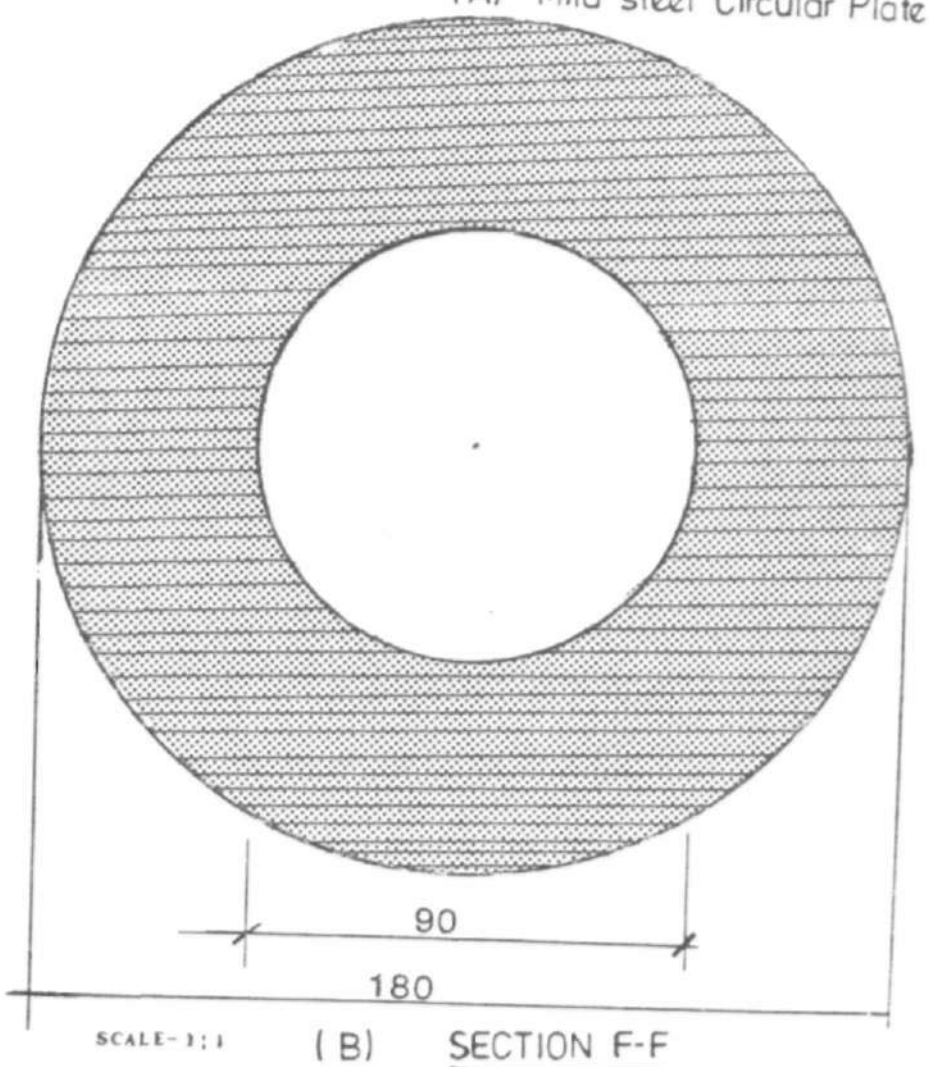
TOP LOADING PLATE AND PISTON

A 65



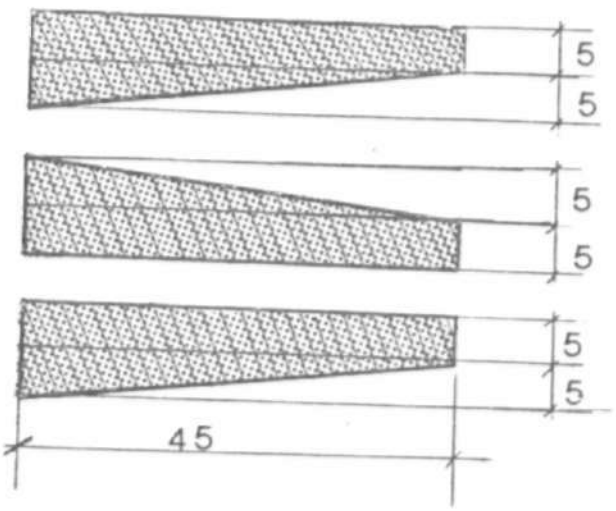
SCALE 1:1

(A) Mild steel Circular Plate Spring

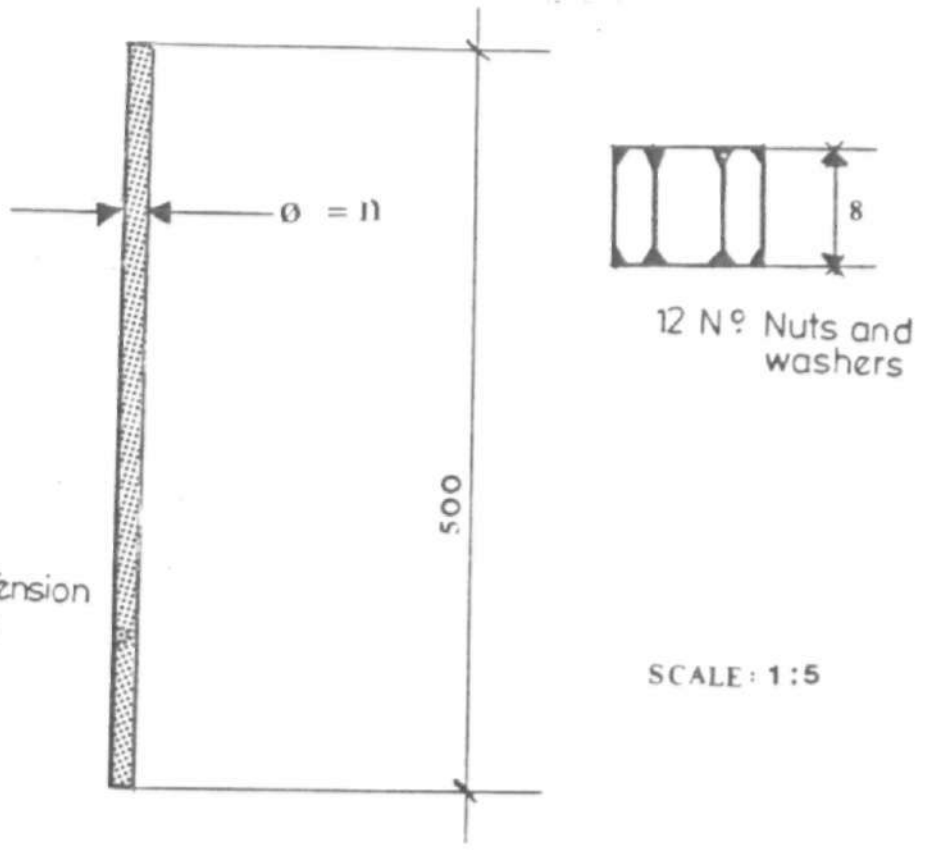


SCALE- 1:1 (B) SECTION F-F

MILD STEEL CIRCULAR PLATE SPRINGS A 66



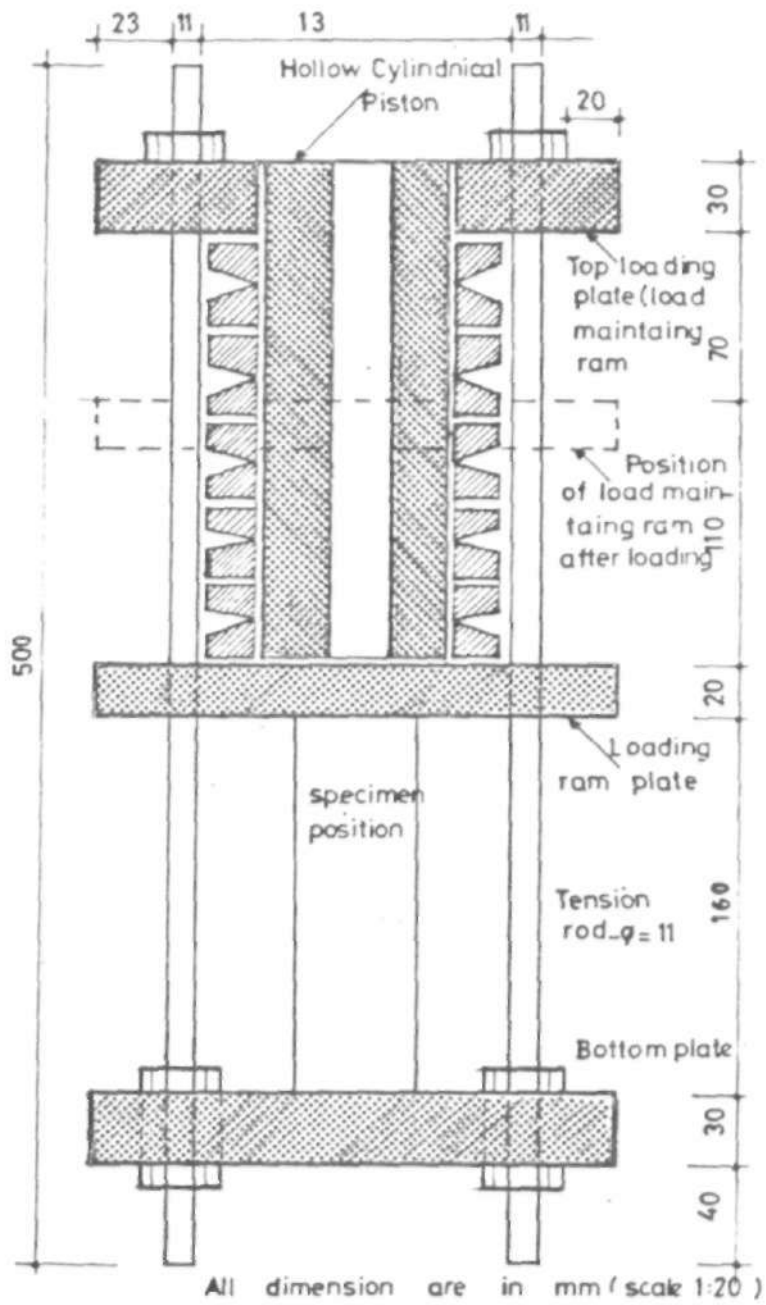
(A) Detail E



SCALE: 1:5

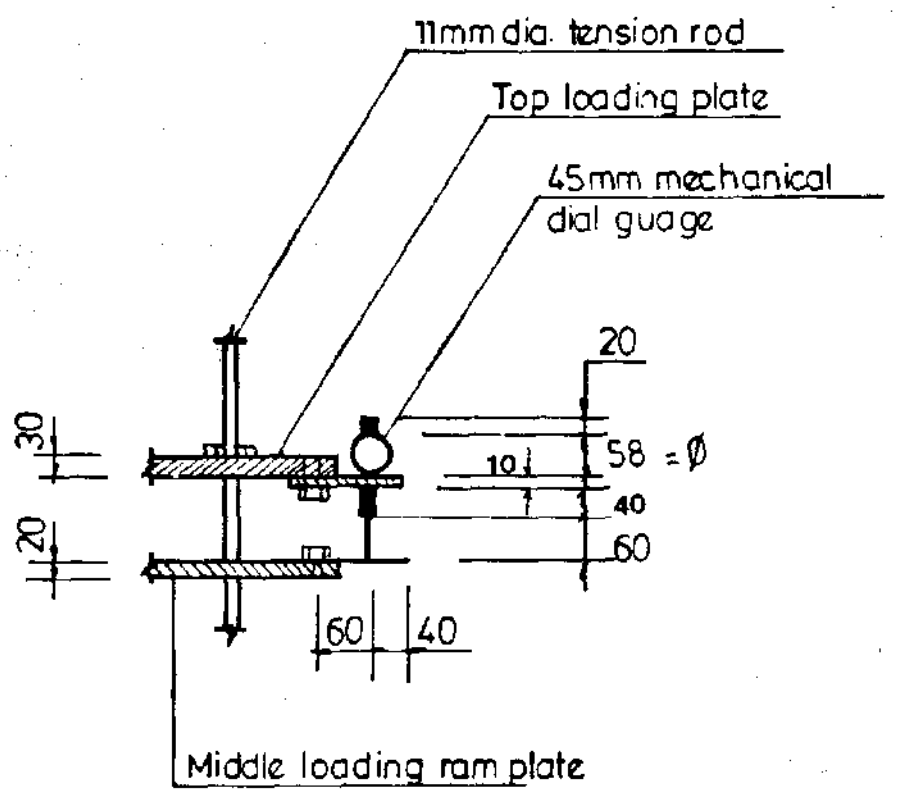
DETAIL 'E', TENSION ROD AND NUT

A 67



ASSEMBLED CREEP FRAME.

A 68



FIXATION OF 45mm MECHANICAL GUAGE TO THE SPRING-LOADED CREEP MACHINE
(scale 1:10)

A 69

A 70

LONG-TERM DEFORMATION MEASUREMENT

Relative Humidity = 50 ± 2 per cent
Temperature = 23 ± 1 °C
Sample Number = 50 CP - 03
Creep machine No. = 01

Serial Number	Time (days)	Deformations mm x 10^{-3}			Strain x 10^{-5}			Average x 10^{-5}
		Point 1	Point 2	Point 3	Point 1	Point 2	Point 3	
1	b/4 loading	0.4+22	0.5+43	0.4+35	-	-	-	-
2	after loading	0.5+16	98	0.5+5	94	55	70	73
3	0.042	22.5	98	7.8	100.5	55	72.8	76.1
4	0.125	23	98	8	101	55	73	76.3
5	0.167	23.2	98	8	101.2	55	73	76.4
6	0.208	23.2	98	8	101.2	55	73	76.4
7	0.250	24	98	9	102	55	74	77
8	0.750	26	98	10	104	55	75	78
9	1	27	98	11	105	55	76	78.7
10	2	29	98	12.5	107	55	77.5	79.8
11	3	30	99	13	108.	56	78	80.7
12	4	0.5+31	0.5+99	0.5+14.8	109	56	79.8	81.6
13	5	31.5	99.2	14.8	109.5	56.2	79.8	81.8
14	6	35	0.6+0	26.8	113	57	91.8	87.3
15	7	36	0.2	27.5	114	57.2	92.5	87.9
16	14	40	3	32	118	60	97	91.7
17	21	43	5	34	121	62	99	94
18	28	46.5	7	37	124.5	64	102	96.8
19	56	58	15	54	136	72	119	109
20	84	65	19	61	143	76	126	115
21	112	71	23	66	149	80	131	120
22	140	88	29	67	166	86	132	128
23	168	94	34	71	172	91	136	133

A 7 1

LONG-TERM DEFORMATION MEASUREMENT

Relative Humidity = 50 ± 2 per cent Temperature = 23 ± 1°C

Sample Number = 50 CP 04 Creep machine No = 2

Serial Number	Time (days)	Deformations mm x 10^{-3}			strains x 10^{-5}			Average x 10^{-5}
		Point 1	Point 2	Point 3	Point 1	Point 2	Point 3	
1	b/4 loading	0.5+13	0.4 + 41	0.3 + 46	-	-	-	-
2	after loading	0.5 + 93	0.5 + 14	0.4 + 0	80	73	54	69
3	0.042	97	18	0.5	84	77	54.5	71.8
4	0.125	98	18.5	0.5	85	77.5	54.5	72.3
5	0.167	98.2	19	1.0	85.2	78	55	72.7
6	0.208	98.5	19	1.0	85.5	78	55	72.8
7	0.250	99	20	1.0	86	79	55	73.3
8	0.750	0.6 + 3	22	1.0	90	81	55	75.3
9	1	3	23.5	1.5	90	82.5	55.5	76
10	2	5	26	2	92	85	56	77.7
11	3	5.5	27.5	2.3	92.5	86.5	56.3	78.4
12	4	7.5	28.5	3.0	94.5	87.5	57	79.7
13	5	9.0	30	3.5	96	89	57.5	80.8
14	6	10.0	30.5	4.0	97	89.5	58	81.5
15	7	10.5	31.2	4.0	97.5	90.2	58	81.9
16	14	16.5	37.5	6.5	103.5	96.5	60.5	86.8
17	21	21	42	10	108	101	64	91
18	28	23.5	45	11.5	110.5	104	65.5	93.3
19	56	39	51	20	126	110	74	103.3
20	84	46	63	26	133	122	80	111.7
21	112	56	70	30	143	129	84	118.7
22	140	63	76	34.8	150	135	88.8	124.6
23	168	68	82	39	155	141	93	129.7

A 7 2

LONG-TERM DEFORMATION.

Relative humidity = 70 ± 4 per cent Temperature = 14°C

Sample Number = 70 CP - 01 Creep stand No. = 8

Serial Number	Time (days)	Deformations mm x 10^{-3}			strains x 10^{-5}			Average x 10^{-5}
		Point 1	Point 2	Point 3	Point 1	Point 2	Point 3	
1	B/4 loading	26+480	0.7+24	0.4 + 96	-	-	-	-
2	after loading	562	0.8+61	0.5+85	82	137	89	102.7
3	0.042	565	62	88	85	138	92	105
4	0.125	570	71.5	90	90	147.5	94	110.5
5	0.167	570	73	90	90	149	94	111
6	0.208	570	73	90	90	149	94	111
7	0.250	570	74	90.5	90	150	94.5	111.5
8	0.750	574	79	93	94	155	97	115.3
9	1	574	80	93.5	94	156	97.5	115.8
10	2	578	85	95	98	161	99	119.3
11	3	580	85	97	100	161	101	120.7
12	4	581	89	0.6+0	101	165	104	123.3
13	5	585	92	3.5	105	168	107.5	126.8
14	6	588	93.5	4	108	169.5	108	128.5
15	7	588	94.5	4	108	170.5	108	128.8
16	14	26 + 589	0.8 + 96	0.6 + 4.5	109	172.0	108.5	129.8
17	21	592	0.9 +4.5	8	112	180.5	112	134.8
18	28	598	7.5	10	118	183.5	114	138.5
19	56	613	29	26	133	205	130	156
20	84	620	47	27	140	223	131	164.7
21	112	623	51	30	143	227	134	168
22	140	636	72	39	156	248	143	182,3
23	168	640	84	42	160	260	146	188.7

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LONG TERM DEFORMATIONS

Relative humidity = 70 ± 4 per cent Temperature = 14°C

Sample Number = 70 CP - 02 Creep stand No. = 2

Serial Number	Time (days)	Deformations mm x 10^{-3}			strains x 10^{-5}			Average x 10^{-5}
		Point 1	Point 2	Point 3	Point 1	Point 2	Point 3	
1	b/4 loading	0.9 + 45	0.7 + 0	30 + 95	-	-	-	-
2	after loading	1 + 32	71	162	87	71	67	75
3	0.042	36	76	165	91	76	70	79
4	0.125	39	78	168	94	78	73	81.7
5	0.167	40	78	169	95	78	74	82.3
6	0.208	40	78	169	95	78	74	82.3
7	0.250	40.5	78.5	169	95.5	78.5	74	82.7
8	0.750	44.8	80	170	99.8	80	75	84.8
9	1	45	80.5	170	100	80.5	75	85.2
10	2	48	83	173	103	83	78	88
11	3	50	84	174	105	84	79	90.3
12	4	53	88	178	108	88	83	93
13	5	56	91	180	111	91	85	95.7
14	6	58	92	180	113	92	85	96.7
15	7	58	92.5	181	113	92.5	86	97.2
16	14	59	93	182	114	93	87	98
17	21	65	97	186	120	97	93	103.3
18	28	68	99.5	190	123	99.5	95	105.8
19	56	87	0.8+15	205	142	115	110	122.3
20	84	94	16	210	149	116	115	126.7
21	112	1.1+ 0	20.5	214	155	120.5	119	131.5
22	140	16	27	224	171	127	129	142.3
23	168	+23	31	230	178	131	135	148

A 7 4

LONG - TERM SHRINKAGE DEFORMATIONS

Relative humidity = 50 ± 2 per cent

Temperature = 23 ± 1 °C

Serial Number	Time	Deformations mm x 10^{-2}		Shrinkage x 10^{-5}		Average x 10^{-5}
		50 SK - 01	50 SK - 02	50 SK - 01	50 SK - 02	
1	Initial	-	-	-	-	-
2	reading	7 + 51.2	8 + 20	0	0	0
3	0.042	51.2	20	0	0	0
4	0.125	51.2	20	0	0	0
5	0.167	51.2	20	0	0	0
6	0.208	51.2	20	0	0	0
7	0.250	51.2	20	0	0	0
8	0.750	51.2	20	0	0	0
9	1	51.2	20	0	0	0
10	2	51.2	20	0	0	0
11	3	51.2	20	0	0	0
12	4	51.2	20	0	0	0
13	5	51.2	20	0	0	0
14	6	51.2	20	0	0	0
15	7	51.2	20	0	0	0
16	14	51.2	20	0	0	00 0
17	21	51.3	20	0.7	0	0.35
18	28	51.2	20	0	0	0
19	56	51.2	20	0	0	0
20	84	51.2	20	0	0	00
21	112	51.2	20	0	0	0
22	140	51.0	20	-1.3	0	-0.65
23	168	51.0	20	-1.3	0	-0.65

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LONG-TERM SHRINKAGE DEFORMATIONS

Relative humidity = 70 ± 4 per cent Temperature = 14°C

Sample Number = 70 SK - 01

Serial Number	Time (days)	Deformations mm x 10^{-3}			Shrinkage strain x 10^{-5}			Average x 10^{-5}
		Point 1	Point 2	Point 3	Point 1	Point 2	Point 3	
1	Initial	-	-	-	-	-	-	-
2	Reading	10 +240	8 + 420	4 + 360	-	-	-	-
3	0.042	240	420	360	0	0	0	0
4	0.125	240	420	360	0	0	0	0
5	0.167	240	420	360	0	0	0	0
6	0.208	240	420	360	0	0	0	0
7	0.250	240	420	360	0	0	0	0
8	0.750	240	420	360	0	0	0	0
9	1	235	420	360	-5	0	0	-1.7
10	2	235	420	360	-5	0	0	-1.7
11	3	240	420	360	0	0	0	0
12	4	10+240	8+420	4 + 360	0	0	0	0
13	5	240	420	360	0	0	0	0
14	6	240	420	360	0	0	0	0
15	7	242	420	360	2	0	0	0.7
16	14	238	415	360	-2	-5	0	-2.3
17	21	238	415	360	-2	-5	0	-2.3
18	28	240	415	355	0	-5	-5	-3.3
19	56	243	420	360	3	0	0	1.0
20	84	240	416	360	0	-4	0	-1.3
21	112	238	412	359	-2	-8	-1	-3.6
22	140	240	413	360	0	-7	0	-2.3
23	168	238	410	350	-2	-10	-10	-7.3

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LONG - TERM SHRINKAGE DEFORMATIONS

Relative humidity = 70 + 4 per cent
Temperature = 14°C
Sample Number = 70 Sk - 02

Serial Number	Time (days)	Deformations mm x 10^{-3}			strains x 10^{-5}			Average x 10^{-5}
		Point 1	Point 2	Point 3	Point 1	Point 2	Point 3	
1	Initial	-	-	-	-	-	-	-
2	Reading	5 +900	0+ 610	7+ 180	-	-	-	-
3	0.042	900	620	180	0	10	0	3.3
4	0.125	905	622	180	5	12	0	5.7
5	0.167	905	622	180	5	12	0	5.7
6	0.208	905	622	180	5	12	0	5.7
7	0.250	905	622	180	5	12	0	5.7
8	0.75	905	622	180	5	12	0	5.7
9	1	905	622	180	5	12	0	5.7
10	2	905	622	180	5	12	0	5.7
11	3	905	622	180	5	12	0	5.7
12		5+ 905	0.+622	7+ 180	5	12	0	5.7
13	5	905	622	180	5	12	0	5.7
14	6	905	622	180	5	12	0	5.7
15	7	910	625	185	10	15	5	10
16	14	910	620	180	10	10	0	6.7
17	21	910	622	180	10	12	0	7.3
18	28	905	622	180	5	12	0	5.7
19	56	910	630	185	10	20	5	11.7
20	84	910	622	180	10	12	0	7.3
21	112	906	620	179	6	0	-1	1.7
22	140	908	620	180	8	0	0	2.7
23	168	900	617	175	0	7	-5	0.70

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AVERAGE SHRINKAGE MEASUREMENT FOR 70 ± 4 PER CENT RELATIVE HUMIDITY

Serial	Time (days	Shrinkage deformation $\times 10^{-5}$		Average Shrinkage $\times 10^{-5}$
		705 k-01	70 SK-02	
1	-	-	-	-
2	-	-	-	-
3	0.042	0	1.0	0
4	0.125	0	3.3	1.7
5	0.167	0	5.7	2.9
6	0.208	0	5.7	2.9
7	0.250	0	5.7	2.9
8	0.750	0	5.7	2.9
9	1	- 1.7	5.7	2.0
10.	2	- 1.7	5.7	2.0
11	3	0	5.7	2.9
12	4	0	5.7	2.9
13	5	0	5.7	2.9
14	6	0	5.7	2.9
15	7	0.7	10	5.4
16	14	-2.3	6.7	2.2
17	21	-2.3	7.3	2.5
18	28	-2.3	5.7	1.7
19	56	1.0	11.7	6.4
20	84	-1.3	7.3	3.0
21	112	-3.6	1.7	-0.95
22	140	-2.3	2.7	0.2
23	168	-7.3	+0.70	-3.3

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